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THE PRACTICAL DESIGN OF STEEL-FRAMED SHEDS.

THE
PRACTICAL DESIGN OF STEEL-FRAMED SHEDS.

BY

ALBERT S. SPENCER.

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PREFACE.

The object of this book is that of providing for the student and the designer a guide to the practical design of shed framework and foundations.

Although there are many excellent treatises which deal wholly, or in part, with the same subject, the justification for this book lies in the fact that the extensive experience of the author has convinced him that too much attention is given in such treatises, and also in the instruction provided at our technical colleges, to the design of the various units without proper regard to the units as a whole. The effect of this is clearly visible in present drawing-office practice, and it is hoped that this book will focus attention to the advisability of preparing designs in a more coherent and consistent manner, due regard being paid to the stability of the structure as a whole.

The author has presumed of the reader an intimate knowledge of "The Theory of Structures" and of practical designing, and therefore has confidence in recommending wind forces and allowable stresses which—although extensively used in competitive designing—may not meet with the entire approval of conservative designers.

In preparing that portion of the book dealing with foundations, the author has appreciated the lack of information open to the designer, and has therefore preferred to offer a rational solution of the difficulty of providing fixity for stancheon bases which, with due care, may be adopted with safety, rather than follow the orthodox procedure of ignoring the problem, or of offering a solution which would be rejected as commercially impracticable.

The author takes this opportunity of thanking the editor of *The Engineer* for permission to reprint the subject matter of the first chapter, and also of acknowledging the assistance rendered to him by his colleagues in the preparation of the various tables and illustrations. The book has been carefully scrutinized for errors, and if any are found by readers an intimation of such would be gratefully acknowledged.

ALBERT S. SPENCER.

MANCHESTER,
October, 1914.

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THE PRACTICAL DESIGN OF STEEL-FRAMED SHEDS.

CHAPTER I.

THE RESISTANCE OF STEEL-FRAMED SHEDS TO WIND FORCES.

Introduction.—The subject of this chapter is one which does not receive proper attention on the part of most engineers engaged in the designing of structural steelwork, especially of the nature of that under consideration. There are many treatises on the "Theory of Structures," which, whilst dealing fairly conclusively with the design of units, very rarely attempt to treat the subject in a coherent manner, i.e. the manner which closely approximates to actual conditions. The same inadequate attention is also common in technical instruction, the effect being that engineers are trained in methods, the application of which give unsuitable results, hence the present alienation of theory from practice.

The designing of sheds is left almost entirely to the manufacturers of such structures. This has led to the existence of certain recognised standards, common throughout what is known as "the light constructional trade," such standards—viewed from the standpoint of theoretical requirements—being ridiculously inadequate and inconsistent. Sheds are at the present time habitually designed and manufactured, portions of which are only capable of resisting a horizontal wind pressure as low as 5 to 7 lbs. per square foot of vertical surface, whilst—in the same structure—portions are deliberately designed for a wind pressure of 30 to 40 lbs. per square foot, the stress per square inch being the same in each case. Further, secondary stresses are often completely ignored, and the foundations provided are totally inadequate, in spite of the fact that the stability of the whole structure finally depends upon proper fixity at the base.

To the skilful and conscientious engineer competitive designing under these conditions is impossible. Knowing full well that the design chosen for acceptance will in all probability be that which accompanies the lowest tender, he must either take the responsibility of a design he cannot rationally uphold, or, on the other hand, be sponsor for one which will be competitively unsuccessful.

It may be pointed out that, such structures having proved themselves fully capable of resisting the wind and other forces to which they are subjected, there is sufficient justification for their continued use, but the fact that portions may actually be stressed up to the point of the elastic limit of the material necessitates for public safety alone a more generous distribution of material. Further, the wide range of stresses set up in the various members calls for greater consistency in design.

The various standards used were in many instances the result of practical demonstration rather than of theoretical consideration. It is no new departure in engineering history for theory to follow practice—instance the theory of reinforced concrete—nor is it uncommon for theory to modify, or even to lead to the disuse of, certain preconsidered methods. Undoubtedly the theory of reinforced concrete led to the discovery of uneconomic uses of materials in various systems in common use, and similar changes are necessary in structural design if the benefits which arise from the judicious use of theoretical knowledge are to be utilised.

The application of the theory of structures hereafter described—in this country especially—has been very seldom used, partly owing to lack of guidance from recognized authorities, but also to the incompatibility of results obtained by following the usual theoretical methods with those commonly arrived at by practical experience.

Wind Pressure.—According to the more recent authors of books on structural design, the horizontal wind pressure to be allowed for on a large area such as that existent in a building is usually given at from 20 to 40 lbs. per square foot of vertical surface, the amount varying with the height of the particular area from ground-level, and also with its position relative to adjacent sheltering buildings. There has undoubtedly been a tendency in recent years towards a decrease in the allowance to be made for the effect of wind pressure on structures, with which movement the writer is in entire agreement, such opinion being based on experience and observation, and there is still further necessity for enlightenment on the subject.

Three years ago the writer communicated with the Director of the Meteorological Office, London, also with the Director of the Observatory at the Manchester University, with a view to obtaining the maximum recorded wind pressure, with the following results:—

Meteorological Office, London.—"The maximum wind velocity which has been recorded by an anemometer for which records are received at this office is 103 miles per hour in a gust at Pendennis Castle, Falmouth; converted into pressure (pounds per square foot) by the formula $P = \cdot 003\ V^2$, this corresponds with a pressure of 32 pounds per square foot."

Manchester University.—"The maximum wind velocity record in Manchester by anemometer since 1896 is 50 miles per hour, which is equal to a pressure of 8 pounds per square foot; the maximum gust velocity would probably be 70 miles per hour, which is equal to about 16 pounds per square foot."

The pressure of wind acting on a large area varies according to laws not yet fully understood. It is a product—variable—of the velocity, and the effect on a large surface is less marked than on a small surface.

Professor Adams suggests allowances which are given in Table 1, Plate 1/A.

The data used in competitive designing by the writer for sheds and similar structures are as follows :—

For proportioning roof framework.—Allowance made for a horizontal wind pressure of 30 to 40 lbs. per square foot of vertical surface, taken as acting normally to the roof surface and reduced to its normal equivalent by the coefficients given in Table 2, Plate 1/A.

Allowable Stresses.—For main framing such as principals, safe stress per square inch equal to one-fourth the ultimate stress; for secondary framing, i.e. purlins and windbracing, safe stress per square inch equal to one-third to one-fourth the ultimate stress.

For proportioning side and end framing.—Allowance made for a horizontal wind pressure of 20 to 25 lbs. per square foot of vertical surface in addition to the lateral force transmitted from roof to stancheons, such force being the horizontal component of a wind pressure of 20 to 32 lbs. per square foot of vertical surface reduced to its normal equivalent and distributed to the windward and leeward stancheons in the manner hereafter described. Allowable stresses taken at one-third to one-fourth the ultimate stress per square inch.

In proportioning foundations provision should be made for the necessary fixity required by the stancheons to resist the wind forces acting on them as before described.

For the foundation bolts—if any—the allowable stress may be taken at one-third to one-fourth the ultimate stress per square inch.

Types of Sheds.—The various types of sheds may be classified for the purpose of this chapter according to the degree of fixity provided for the side stancheons at their bases and caps, and also to the use or otherwise of side stays, which act as supports to the stancheons. Primarily, there are four types of stancheons, viz. :—

Type 1.—Where the stancheons have a small degree of fixity at their caps and bases, which, being infinitesimal, may be ignored, and the connections assumed to act as pin-joints.

Type 2.—Where the stancheons are considered to be perfectly fixed at their bases, and have a slight amount of fixity at their caps, the latter being ignored and subject to the assumption referred to in Type 1.

Type 3.—Where the stancheons are assumed as pin-jointed at their caps and bases, and are provided with supports (side stays).

Type 4.—Where the caps of the stancheons are assumed to be pin-jointed, the bases perfectly fixed, and stays—acting as supports—are provided.

Type 1, although in common use, is unstable and unsuitable for use unless the shed is constructed with several bays, the lateral forces distributed over the entire width, and the end connections of the stancheons designed to give the necessary fixity. Owing to the impracticability of perfectly fixing the connection of the stancheons to the roof principals without the use of side stays, this type of shed is not recommended, and will not be further considered.

Type 2 is generally adopted where a crane gantry is to be provided, which prevents the use of side stays, unless they are placed clear of the crane. Such procedure necessitates an increased height of shed, which is uneconomical and often undesirable.

Type 3 should be used where the nature of the ground is such that the amount of resistance at the foundation must be kept to a minimum.

Type 4 is the most suitable and economical arrangement, and is recommended for general use where the earth is capable of offering suitable resistance.

The various types are outlined on Plate 1/A.

Vertical Position of Reaction Point.—The reaction point at each side and for all types will coincide with the point of contraflexure in the side stancheon, and the position of this point will vary with the degree of fixity.

Type 2.—The connection of the stancheon to the principal will in practice partially fix the end of the stancheon, and the point of contraflexure will be at some point adjacent to this connection. To simplify the calculation of the stresses the connection may be considered as pin-jointed, in which case the reaction point will be at the centre of the connection, i.e. the intersection of the lines of resistance of the members acting at this point.

Type 3.—The degree of fixity at the base being negligible, the connection here may be assumed as acting as a pin-joint, and the point of contraflexure as situated at the centre of the base of the stancheon.

Type 4.—The point of contraflexure will be at some position between the foot of the stay and the base, such position depending upon the distance of the stay connection from the cap and base of the stancheon and the degree of fixity at these extremities. In practice this point may be assumed as midway between the foot of the stay and the ground-level, which assumption is on the side of safety.

Distribution of Lateral Forces.—The lateral force acting on the roof is transmitted to the windward and leeward stancheons by the roof principal, which also acts as the medium for that portion of the force acting on the windward side and resisted by the leeward stancheon.

Type 2.—For this type the stancheons and principals should be designed as separate units. The sections of the windward and leeward stancheons being the same, the deflection at their caps and also their bending moments at their bases may be assumed to be equal; therefore the lateral forces acting on each will also be equal, the amount being in each case one-half the total horizontal force acting on the shed. The reaction points of the principals may be considered as both equally free to move, and therefore the horizontal components of the roof wind force will be equal at each reaction point.

Types 3 and 4.—For these types the stancheons and principals will act in a coherent manner. The reaction points are assumed free to move equally, and the horizontal components of side and roof wind forces will therefore also be equal in effect.

For all types the amount of the vertical component of the wind and also of the dead load transmitted to each reaction point will be inversely proportional to the distance of their resultants from the reaction considered.

It should be noted that the amount of side wind force transmitted to the leeward stancheon may set up compressive stresses in the bottom chord of the principal, and the latter should be provided with the necessary resistance to side flexure to reduce the effective length as a strut. This resistance may be obtained by the use of longitudinal ties placed at suitable positions along the chord, which ties should have proper anchorage at the gables.

Distribution of End Wind Force.—The wind acting on the gable is transmitted by the sheeting rails and intermediate stancheons to their respective points of support, i.e. the ground line, the sides of the building, and the roof, and, as finally the whole of the resistance is derived from the foundations, provision should be made accordingly. That portion transmitted to the roof is by means of bracing, placed either in the plane of the roof or of the bottom chords of principals (or both), transferred to the sides of the buildings, whence, in addition to that taken direct by the sheeting rails, it is transmitted to the foundations by means of bracing in the sides and ends of the building.

Practical application to the design of a steel-framed shed 120 ft. long × 60 ft. span × 20 ft. high to eaves level, covered externally with corrugated sheeting and capable of withstanding a horizontal wind force of 32 lbs. per square foot for the roof and 20 lbs. for the side. Centres of stancheons and principals to be 12 ft., angle of roof 24½°.

Comparing Types 2, 3, and 4, the only variation in the stresses will be those for the side stancheons and their foundations, the roof principals, and the stays—if any used.

Side Stancheons, Type 2.—The lateral forces are resisted at the foundations by two forces acting as a "couple." The base of the stancheon will be embedded in concrete, and therefore the positions of the two forces may be assumed to coincide with the centre of the resultant pressure of the earth on the windward and leeward sides of the foundation. The centre of pressure for the leeward and windward sides will be one-third and two-thirds the depth of the concrete from the ground-level respectively. The maximum bending moment will occur at the plane of the leeward resultant, but for simplicity will be assumed as at the ground-level, the amount—ignoring effect of vertical load—being found as follows :—

SYMBOLS

A = angle of roof.
AS = angle of stay to stancheon.
B = length of roof slope (for A = 24½°, B = ·55C).
BM = bending moment.
C = span of roof (distance between centres of stancheons).
D = position—relative to windward stancheon—of intersection of resultant NP with horizontal line through principal reactions (R^P), (for A = 24½°, D = ·3C).
D^1 = do. do. through reactions (R^T) of transverse frames = $\cdot 3C + \cdot 4557S^2$.
F = stress per square inch.
L = leverage of HN acting at R^T or R^P about point of maximum BM.
L^1 = leverage of THP acting at R^T or R^P about point of maximum BM.
L^2 = leverage of HP^S about R^T or R^P.
HP = horizontal wind pressure.
HP^S = ,, ,, ,, acting on side.
THP = total HP^S on side above reaction point for transverse frame.
PHP = HP^S on portion of side between stay and reaction point of transverse frame.
LHP = portion of HP^S transmitted to leeward stancheon.
RHP = ,, ,, acting at WR^T.
NP = normal wind pressure.

VN = vertical component of NP = cos A × NP (for A = 24½°, VN = ·91 NP).

HN = horizontal component of NP = sine A × NP (for A = 24½°, HN = ·4146 NP).

MS = modulus of section.

MR = moment of resistance.

R = maximum radius of gyration.

R^1 = minimum " " "

R^T = reaction points for transverse frames.

R^P = " " " principals.

S = distance of stay connection from R^T.

S^1 = " " " " " R^P.

S^2 = " " R^T from R^P.

W = weight of roof covering and framing.

W^1 = " " side covering and rails.

VS = vertical reactions due to L^2.

HP = 32 lbs.

NP = 32 × ·56 = 18 lbs.
HN = ·4146 NP = 7½ lbs.
} per square foot of roof surface.

B = length of roof slope = ·55 span = 33 ft.

Area of roof = 33 × 12 = 396 square feet.

HN = 396 × 7½ = 2970 lbs.

L = 20 feet.

Area of side = 20 × 12 = 240 square feet.

HP^S = 240 × 20 = 4800 lbs.

L^2 = 10 feet.

$$BM=\left(\frac{HN}{2}\times L\right)+\left(\frac{HP^S}{2}\times L^2\right)\quad\text{or}$$

$$(1485\times 20)+(2400\times 10)=53{,}700\text{ ft.-lbs. and }644{,}400\text{ inch-lbs.}$$

W = 8 lbs. per square foot of roof surface = 396 × 8 = 3168 lbs.

W^1 = 5 " " " " " side " = 240 × 5 = 1200 "

VN = ·91 NP = 16·38 lbs. per square foot of roof surface = 396 × 16·38 = 6487 lbs. D = ·3C and

Proportion of VN on windward stancheon = ·7 VN = 4541 lbs.
" " W " " " = 3168 "
" " W^1 " " " = 1200 "
Total 8909 "

The side rails may be considered to assist the stancheon in the direction of the least radius of gyration. The maximum stress due to bending moment is at the base, and this stress being the major portion of the whole, the total length of the stancheon need not be taken as the effective length of the strut, but as the connection at the cap approximates to a pin-joint, this adverse effect on the strut will be assumed to balance the advantage due to the position of the maximum bending moment, and the stancheon stress is therefore taken as for a strut, fixed at each end, and with a radius of gyration taken about an axis in a direction parallel to the flanges; this method will keep the compressive stress within safe limits.

Assuming 12″ × 6″ × 44 lbs. RSJ, area = 13 square inches.

$MS = 52{\cdot}5$

$R = 5$

$$\text{Length} = 240 \text{ inches} \therefore \frac{\text{length}}{R} = 48.$$

Safe compressive stress for $\frac{L}{R} = 48$ is 13,340 lbs. per square inch.

Actual compressive stress $= \frac{W}{A} + \frac{BM}{MS} = \frac{8909}{13} + \frac{644{,}400}{52{\cdot}5} = 12{,}959$ lbs. per square inch (which is within the limit of safe stress).

Side Stancheons, Type 3.—The maximum bending moment will occur at the level of the intersection of the centre of resistance of the side stay with that of the stancheon, say, 5 ft. down from cap.

The total lateral forces acting on both stancheons are:—

$HN + THP = 2970 + 4800 = 7770$ lbs.

L and $L^1 = 15$ ft.

Total positive moment $= 7770 \times 15' = 116{,}550$ ft.-lbs.

Total negative moment $= (15'\text{-}0'' \times 12'\text{-}0'' \times 20) \times 7'\text{-}6'' = 27{,}000$ ft.-lbs.

$\therefore$ BM for each stancheon $= \frac{116{,}550 - 27{,}000}{2} = 44{,}775$ ft.-lbs. or 537,300 in.-lbs.

The vertical forces acting on the stancheons are those due to the weight of the roof (W) and sides (W^1), the vertical component (VN) of the roof wind force, and the vertical forces (VS) due to the leverage of the side wind force.

$D^1 = {\cdot}3C + {\cdot}4557S^2 = 18 \text{ ft.} + 9{\cdot}114 \text{ ft.} = 27{\cdot}114 \text{ ft. or } {\cdot}45C.$

The vertical forces for the windward and leeward stancheons are:—

WINDWARD STANCHEON			LEEWARD STANCHEON		
$VN =$	$\cdot 55\,VN$	$= +3568$	$VN =$	$\cdot 45\,VN$	$= +2919$
$W =$		$+3168$	$W =$		$+3168$
$W^1 =$		$+1200$	$W^1 =$		$+1200$
$VS =$	$\frac{HP^S \times L^2}{S} = \frac{4800 \times 10'}{60'}$	$= -800$	$VS =$		$+800$
	Total	$+7136$ lbs.		Total	$+8087$ lbs.

Assuming 12″ × 5″ × 39 lbs. RSJ, $MS = 43{\cdot}48$, $R = 4{\cdot}77$, area $= 11{\cdot}47$, and $\frac{L}{R} = 38$, safe compressive stress $= 13{,}855$ lbs. per square inch.

Actual compressive stress $=\frac{W}{A}+\frac{BM}{MS}=\frac{8087}{11\cdot47}+\frac{537,300}{43\cdot48}=$ 13,064 lbs. per square inch.
(W = total downward force on stancheon.)

Side Stancheons, Type 4.—The point of contraflexure is assumed as midway between the intersection of the stay with the stancheon and the ground-level, the former being the position of the maximum bending moment above the reaction point, and the latter the position below the reaction point.

HN = 2970 lbs.
THP = 3000 „
L and $L^1 = 7'\text{-}6''$.

Positive moment for both stancheons $=(HN+THP)\times L^1=(2970+3000)\times 7\frac{1}{2}'=44,775$ ft.-lbs.
Amount of load acting negatively $(PHP)=7'\text{-}6''\times 12'\times 20=1800$ lbs.
Negative moment $=1800\times 3\frac{3}{4}'=6750$ ft.-lbs.
$\therefore$ BM at stay for each stancheon $=\frac{44,775-6750}{2}=$ 19,012 ft.-lbs. or 228,144 inch-lbs.
BM at ground-level $=19,012+6750=25,762$ ft.-lbs. $=309,144$ inch-lbs.
$D^1=\cdot3C+\cdot4557S^2=18$ ft. $+5\cdot7$ ft. $=23\cdot7$ ft. or $\cdot4C$.

The vertical forces on each stancheon are :—

WINDWARD STANCHEON			LEEWARD STANCHEON		
VN =	·6 VN	= +3892	VN =	·4 VN	= +2595
W =		+3168	W =		+3168
W^1 =		+1200	W^1 =		+1200
VS =	$\frac{3000\times 6\frac{1}{4}'}{60'}$	− 313	VS =		+ 313
	Total	+7947 lbs.		Total	+7276 lbs.

Assuming $8''\times 5''\times 28$ lbs. RSJ area $=8\cdot24$ sq. inches and $MS=27\cdot6$.
Actual compressive stress $=\frac{W}{A}+\frac{BM}{MS}=\frac{7947}{8\cdot24}+\frac{309,144}{27\cdot6}=$ 12,164 lbs. per square inch.
As the allowable stress is 14,560 lbs. per square inch, the actual stress is within the safe limit.

It should be noted that as the maximum radius of gyration has been used to determine the safe stress, it is necessary that the positions of the side rails should be such as to assist the compression flange to resist side flexure.

Roof Principals and Transverse Frames.—The forces acting on the principals and transverse frames are :—

W = the weight of the roof framework and covering.
W^1 = „ „ „ „ side „ „ „ at each side.
NP = the normal wind force.
HP^S = wind pressure on side of shed.

Force W^1 only acts on the transverse frames.

The most satisfactory method for determining the stresses in the various members is the graphical method. By the adoption of a fixed angle (A) for the roof slope (24½°) force constants may be determined, the use of which will greatly simplify the calculations for the reactions.

FORCE CONSTANTS FOR REACTIONS. A=24½°.

Type 2, Principals.

W : Amount of W at WRP$=\frac{W}{2}$; at LRP$=\frac{W}{2}$

NP : Resolve into components HN and VN thus :—

$$\left.\begin{array}{l}HN = \text{Sine } A \times NP = \cdot 4146\ NP\\ VN = \text{Cos } A \times NP = \cdot 91\ \ \ \ NP\end{array}\right\}\begin{array}{l}\text{for principals and}\\ \text{transverse frames.}\end{array}$$

Amount of HN at WRP$=\cdot 2073$ NP ; at LRP$=\cdot 2073$ NP

„ „ VN „ „ $=\cdot 637$ NP ; „ „ $=\cdot 275$ NP

HP^S : Amount of HP^S at WRP$=\cdot 25\ HP^S$; at LRP=nil.

Types 3 and 4, Transverse Frames.

W : Amount at WRT$=\frac{W}{2}$; at LRT$=\frac{W}{2}$

W^1 : Amount at WRT$=W^1$; at LRT$=W^1$

NP : „ of HN at WRT$=\cdot 2073$ NP ; at LRT$=\cdot 2073$ NP

The proportion of VN is inversely proportional to the distance D^1 from either reaction point (RT), and as

$$D^1 = \cdot 3C + \cdot 4557S^2, \text{ then}$$

Amount of VN at WRT$=\frac{C-(\cdot 3C+\cdot 4557S^2)}{C}\times VN$; at LRT the amount will be the remainder of VN

$$HP^S : \text{ Amount at LRT} = \frac{HP^S - RHP\ ;}{2}$$

$$\text{at WRT} = \frac{HP^S - RHP}{2} + RHP$$

In addition to the above forces there is an additional force set up at each support for the transverse frames, such force being due to the leverage of the side wind force about the reactions. These forces are equal in magnitude, but are of a different nature, that at the windward side being negative and that at the leeward side positive.

$$HP^S : \left.\begin{array}{l}\text{negative effect at WRT}\\ \text{positive}\ \ \ ,,\ \ \ ,,\ \ \text{LRT}\end{array}\right\} = \frac{HP^S \times L^2}{C}$$

Note.—For Type 4, HP^S referred to above equals THP

CALCULATIONS FOR REACTIONS.

Type 2, Principals.

Total upward reaction at $\text{WRP}=\frac{\text{W}}{2}+\cdot637\ \text{NP}=3168+4540$ $=7708$ lbs.

„ „ „ „ $\text{LRP}=\frac{\text{W}}{2}+\cdot275\ \text{NP}=3168+1945$ $=5113$ lbs.

„ horizontal „ „ $\text{WRP}=\cdot2073\ \text{NP}=1477$ lbs.

„ „ „ „ $\text{LRP}=\cdot2073\ \text{NP}+\cdot25\ \text{HP}^{\text{S}}$ $=1477+1200=2677$ lbs.

Type 3, Transverse Frames.

Total upward reaction at $\text{WRT}=\frac{\text{W}}{2}+\text{W}^1+\left[\frac{\text{C}-(\cdot3\text{C}+\cdot4557\text{S}^2}{\text{C}}\times\text{VN}\right]$ $-\frac{\text{HPS}\times\text{L}^2}{\text{C}}=3168+1200+3568-800=7136$ lbs.

Total upward reaction at $\text{LRT}=\frac{\text{W}}{2}+\text{W}^1+\left[\frac{(\cdot3\text{C}+\cdot4557\text{S}^2)}{\text{C}}\times\text{VN}\right]$ $+\frac{\text{HP}^{\text{S}}\times\text{L}^2}{\text{C}}=3168+1200+2919+800=8087$ lbs.

Total horizontal reaction at $\text{WRT}=\cdot2073\ \text{NP}+\frac{\text{HP}^{\text{S}}-\text{RHP}}{2}+\text{RHP}$ $=1477+1500+1800=4777$ lbs.

Total horizontal reaction at $\text{LRT}=\cdot2073\ \text{NP}+\frac{\text{HP}^{\text{S}}-\text{RHP}}{2}$ $=1477+1500=2977$ lbs.

Type 4, Transverse Frames.

Total upward reaction at $\text{WRT}=\frac{\text{W}}{2}+\text{W}^1+\left[\frac{\text{C}-(\cdot3\text{C}+\cdot4557\text{S}^2)}{\text{C}}\times\text{VN}\right]$ $-\frac{\text{HP}^{\text{S}}\times\text{L}^2}{\text{C}}=3168+1200+3892-313=7947$ lbs.

Total upward reaction at $\text{LRT}=\frac{\text{W}}{2}+\text{W}^1+\left[\frac{\cdot3\text{C}+\cdot4557\text{S}^2}{\text{C}}\times\text{VN}\right]$ $+\frac{\text{HP}^{\text{S}}\times\text{L}^2}{\text{C}}=3168+1200+2595+313=7276$ lbs.

Total horizontal reaction at $\text{WRT}=\cdot2073\ \text{NP}+\frac{\text{HP}^{\text{S}}-\text{RHP}}{2}+\text{RHP}$ $=1477+1050+900=3427$ lbs.

Total horizontal reaction at $\text{LRT}=\cdot2073\ \text{NP}+\frac{\text{HP}^{\text{S}}-\text{RHP}}{2}$ $=1477+1050=2527$ lbs.

The above reactions will enable the force diagram to be completed from which the stress diagram may be drawn in the usual manner, as shown on Plates 1/B, 1/C, and 1/D, for Types 2, 3, and 4 respectively. The amount and nature of the stresses in the various members being known, the sections may be determined according to the following conditions :—

Allowable tensile stress : 7½ tons per square inch of *net* section.

Allowable compressive stress : 6½ tons per square inch (of gross section) reduced by strut formula to the allowable value for $\frac{L}{R}$, the latter being limited to the ratio $\frac{160}{1}$.

If the ends of the struts are free to move in a direction equal to, or at right angles to, the plane of the principal, the end should, of course, be considered as free (pin-jointed) and the stress determined accordingly.

The amount and nature of the stresses, the condition of the ends of the struts, and the sections required are figured with the diagrams.

Owing to the low wind pressures and factors of safety recommended for use, it is advisable that secondary stresses due to eccentric arrangements of intersections of members and of arrangements of rivets or bolts, should be avoided as far as practicable—the neglect of which is a common fault in present-day practice. Further, if the location of the purlins is such that flexural stresses are set up in the rafters, such stresses should be added to the principal stresses, and the rafter designed accordingly.

Side Rails.—The forces acting on the side rails are those due to wind pressure and to the weight of the rails and side covering. The rails are generally spaced at 6′-6″ centres, and will be thus considered. Therefore :—

Horizontal wind pressure (HP^S) on side $=20$ lbs. per square foot.
Area of side supported $=12'\text{-}0'' \times 6'\text{-}6'' = 78$ square feet.
Total HP^S on rail $=78 \times 20 = 1560$ lbs.

$$BM = \frac{WL}{8} = \frac{1560 \times 144''}{8} = 28{,}080 \text{ inch-lbs.}$$

$MR = BM = 28{,}080.$
$MS = MR \div F$, and as $F = 22{,}400$ lbs., then

$$MS = \frac{28{,}080}{22{,}400} = 1{\cdot}25$$

Nearest British Standard Joist Section is 4″ × 1¾″ × 5 lbs. RSJ, with a maximum $MS = 1{\cdot}83$.

$$\therefore \text{ Actual stress} = \frac{28{,}080}{1{\cdot}83} = 15{,}344 \text{ lbs. per square inch.}$$

Assuming the 4″ × 1¾″ rail is supported at its centre by means of a sag rod, then the stress due to the vertical load will be :—

Weight of side sheeting = 6′-0″ × 6′-6″ × 3 lbs. = 117 lbs.
„ „ rail = 6′-0″ × 5 lbs. = 30 „
147 „

$$BM=\frac{WL}{8}=\frac{147\times72''}{8}=1323 \text{ inch-lbs.}$$

Minimum MS = ·22.

$$\text{Stress per square inch}=\frac{1323}{\cdot22}=6013 \text{ lbs.}$$

Total stress per square inch = 15,344 + 6013 = 21,357 lbs., which is within the safe limit of stress.

Roof Purlins.—The forces acting on the purlins are those due to the wind pressure, to the weight of the purlins, and to the weight of the roof covering. The purlins will be spaced at 6′-6″ centres; therefore:—

HP on roof = 32 lbs. per sq. ft.
NP „ „ = 32 × ·56 = 18 lbs. per sq. ft.
Area supported = 12′-0″ × 6′-6″ = 78 sq. ft.
Weight of roof covering, say, 3 lbs. per sq. ft.
Total load on purlin:—
Wind = 78 × 18 = 1404 lbs.
Covering = 78 × 3 = 234 „
Purlin = 12 × 5 = 60 „
Total 1698 „

$$BM=\frac{WL}{8}=\frac{1698\times144''}{8}=30{,}564 \text{ inch-lbs.}$$

$$MS=\frac{30{,}564}{22{,}400}=1\cdot35.$$

∴ use 4″ × 1¾″ × and 5 lbs. RSJ as for side rails.

Owing to the purlins being used at a slight angle to the vertical, there will be a tendency to sag in the direction of the least MS, and it is advisable to provide sag rods similar to those for the sides. The sag rods on the one slope will—if properly connected to the ridge purlin—react against those on the other slope.

In the "light construction trade" the section generally adopted for rails and purlins under similar conditions to those under discussion is a 2½″ × 2½″ × 5/16″ angle, having a MS of ·46, which, although the same weight per foot as the RSJ adopted, is inferior in strength to the extent of 1 : 4.

End Framing.—The end framing will consist of uprights, rails, and rafters, together with the necessary stays, diagonal braces, and sag rods, arranged as shown on Plate 1/A. The determination of the stresses in the various members is somewhat complex, but the sections may be closely approximated, as hereafter described.

Uprights.—

Centres of uprights=15′-0″.
Lengths „ „ =20′-0″, 26′-10½″, and 33′-9″.
H.P. per foot run =15′-0″×20 lbs.=300 lbs.
Total HP on 26′-10½″ length=26⅞×300=8063 lbs.

$$BM=\frac{WL}{8}=\frac{8063\times322\cdot5}{8}=325{,}040 \text{ inch-lbs.}$$

Weight of end covering=26′-10½″×15′-0″×3 lbs.	=1210	lbs.
„ „ rails and bracings	say= 600	„
„ „ stancheon	„ = 868	„
„ „ roof=15′-0″×6′-0″×8 lbs.	= 720	„
Total	3398	„

Assuming a 12″×5″×32 lbs. RSJ, MS=36·66, R=4·8, area=9·4 sq. inches, then

$$\frac{L}{R}=\frac{322}{4\cdot8}=67, \text{ and safe stress}=12{,}430 \text{ lbs. per sq. inch.}$$

$$\text{Actual stress}=\frac{W}{A}+\frac{BM}{MS}=\frac{3398}{9\cdot4}+\frac{325{,}040}{36\cdot66}=9229 \text{ lbs. per sq. inch.}$$

In addition to this stress there is the slight effect due to the lateral wind pressure acting on the first 6′-0″ length of the building, but as there is ample margin between the actual and allowable stress this effect may be ignored.

The vertical load being almost negligible, the 33′-9″ upright may —to expedite the design—be safely approximated from the section calculated above on the principle that "the strength of a RSJ acting as a beam varies directly in proportion to the load and to the span." The load per foot run being about the same, and therefore varying in proportion to the span, then the modulus of section required will equal :—

$$\left(\frac{33'\text{-}9''}{26'\text{-}10\tfrac{1}{2}''}\right)^2\times36\cdot6=57\cdot7.$$

A 12″×6″×44 lbs. RSJ has a modulus of section of 52·5, which, however, will be sufficient.

With regard to the 20′-0″ upright, the section adopted will be that calculated for the 26′-10½″ length, i.e. 12″×5″ RSJ.

End Rails.—The span of the rails will be 15′-0″, and the centres about 6′-6″; as the load per foot run is the same as that for the side, by the pre-stated law the section may be approximated thus :—
MS for side rails=1·83.

$$\text{MS for end rails}=\left(\frac{15}{12}\right)^2\times1\cdot83=2\cdot8.$$

A 4¾″×1¾″×6½ lbs. RSJ with a modulus of section of 2·8 is the nearest suitable section, and will therefore be adopted.

Rafters.—The length of the rafters is 33′-0″, divided up into two spans of 16′-6″. The forces acting on the rafters are :—

(1) End wind force.
(2) Forces due to reactions from stancheons.
(3) Weight of roof, and of end covering and framing.

The stresses set up by these forces are :—

(1) End wind force = 15′-0″ × 1′-11¼″ (average) × 20 lbs. = 600 lbs. The intensity of this force varies from zero at one end of the span to a maximum at the other, and the bending moment may be found by the formula :—

$$\text{BM} = \cdot 1283\ \text{WL, which equals}$$
$$\cdot 1283 \times 600 \times 198 = 15{,}240\ \text{inch-lbs.}$$

(2) This force is the reaction for the 26′-10½″ upright, and will therefore be 4030 lbs., and is resisted by a triangular truss situated in the plane of the roof, the horizontal (compressive) member of which is the rafter.

Span of truss (L) = 33′-0″, and depth (D) = 12′-0″; therefore

$$+\text{stress in rafter} = \frac{\text{WL}}{4\text{D}} = \frac{4030 \times 33}{4 \times 12} = 2770\ \text{lbs.}$$

The roof wind force will not act at the same time as the end wind force, and as the effect of the latter is the greater of the two, the effect of the roof force need not enter into the calculation.

(3) The situation of the roof purlin produces—in addition to the direct compressive stress allowed for later—a bending moment, the amount of which equals :—

$$\text{Roof load (W)} = 6'\text{-}0'' \times 6'\text{-}6'' \times 8\ \text{lbs.} = 312\ \text{lbs., and}$$
$$\text{Approx.}\quad \text{BM} = \frac{\text{WL}}{6} = \frac{312 \times 99}{6} = 5148\ \text{inch-lbs.}$$

The direct compressive stress referred to above is supplemented by that set up by the action of the sag rod, the total being found as follows :—

Approximate load from roof = 312 lbs.
„ „ „ end covering and rails = 723 „
Total = 1035 „

$$+\text{stress in rafter} = \frac{\text{a}}{\text{a} \times \text{Sine A}} \times \frac{1035}{2} = 1249\ \text{lbs.}$$

(a = rise of rafter = 3′-5¼″, A = angle of roof = 24½°—Sine A = ·4146).

Recapitulating, the stresses are :—

(a) Total + stress = 2770 + 1249 = 4019 lbs.
(b) BM Vertical = 5148 inch-lbs.
(c) BM Longitudinal = 15,240 „ „

The total compressive stress about an axis perpendicular to the section is due to stresses (a) and (c), and about an axis horizontal to the section is due to stresses (a) and (b); therefore the section may be found as follows :—

Assuming a 6″×2½″ ⊓ with max. MS = 6·25, min. MS = 1.
max. R = 2·3, min. R = ·73.
Area = 3·5 sq. inches.

$\frac{L}{R}=\frac{99}{\cdot 73}=136$, and safe stress as strut = 7616 lbs. per sq. inch.

Safe flexure + stress = 16,800 lbs. per sq. inch.

Total + stress per sq. inch due to (a) $=\frac{4019}{3\cdot 5}=1148$ lbs.

" " " " (b) $=\frac{BM}{MS}=\frac{5148}{1}=5148$ lbs.

" " " " (c) $=\frac{BM}{MS}=\frac{15{,}240}{6\cdot 25}=2438$ lbs.

The percentage of the actual to the allowable stress =

$$\frac{1148}{7616}\times 100=15\%$$

$$\frac{5148+2438}{16{,}800}\times 100=46\%$$

Total 61%

Therefore the 6″×2½″ ⊓ is ample.

Rafter Stays.—The direct compressive effect of the roof and end load on the stay is similar to that on the rafter, i.e. 1249 lbs. The load being small, the section will be determined by the ratio $\frac{L}{R}$, which should not materially exceed $\frac{160}{1}$; therefore

L = 8′-3″

$R=\frac{L}{160}=\frac{99}{160}=\cdot 62$ and section adopted, say, 3″×3″×¼″ ∟.

Roof Bracing.—The roof bracing is placed in the plane of the roof at each end of the building and takes the form (in plan) of a triangular girder, as shown on Plate 1/A. The functions of the bracing are :—

(1) To transmit the end wind force brought on to the roof by the end stancheons, to the sides of the building.

(2) To give rigidity to the rafter backs in a direction at right angles to the rafter backs.

(3) To prevent distortion due to changes of temperature.

(4) To brace the framework whilst in course of erection.

The whole of the four conditions do not occur simultaneously, and as the most severe combination of stresses will be probably those due to (1) and (2), these only need be considered. The stresses due to (2) will not be a maximum at the same time as those for (1), the reason being that the roof wind will not act at the same time as the end wind—the direction of the former being transverse, whilst that of the latter is longitudinal.

When the end wind is a maximum, the resistance required for function (2) is that necessary to prevent side flexure of the rafter backs, such flexure being caused by the compressive stress set up by the action of the roof principal in transmitting the dead load to the supports.

The effect of function (2) is practically indeterminate, but may be assumed roughly as equal to one-half that for (1), and on this assumption the sections may be approximately determined as follows :—

$$\left.\begin{array}{l}\text{W and W}^1 = \text{wind force at panel points.}\\ \text{A to G} \quad = \text{reference marks for joints.}\end{array}\right\}\text{See Plate A.}$$

Stress in $BF = +W$.
„ „ $CG = +(W + W^1)$.
„ „ $AF = -\left(W + \frac{W}{2}\right) \times \frac{AF}{FB}$.
„ „ $FG = -\frac{(W + W^1) \times AG}{2 \times CG}$.
„ „ $FC = -\left(\frac{W}{2} \times \frac{AG}{CG}\right)$.

The stress in AC and CE due to the outer loads (W) have already been allowed for in designing the gable rafter. Solving the above :—
$W = 4032$ lbs. Add 50% for (2) = 6048 lbs.
$W^1 = 5062$ „ „ „ „ = 7593 „
Stress in $BF = +W = +6048$ lbs.

$$CG = +(W + W^1) = +13{,}641 \text{ lbs.}$$

$$AF = -\left(W + \frac{W^1}{2}\right)\frac{AF}{FB} = -(6048 + 3796) \times \frac{20{\cdot}4}{12} = -16{,}735 \text{ lbs.}$$

$$FG = -\frac{(W + W^1)\ AG}{2 \times CG} = -\frac{(6048 + 7593) \times 40{\cdot}8}{2 \times 24} = -11{,}595 \text{ lbs.}$$

$$FC = -\left(\frac{W}{2} \times \frac{AG}{CG}\right) = -\left(\frac{6048}{2} \times \frac{40{\cdot}8}{24}\right) = -5142 \text{ lbs.}$$

The sections of the members will be :—
BF stress = + 6048 lbs., length of strut — 12′-0″, ends fixed, 2/3″ × 2″ × ¼″ ┐┌
CG „ = + 13,641 „ „ „ — 12′-0″, „ „ 2/3″ × 2½″ × ¼″ ┐┌
AF „ = − 16,735 „ } say 1/3″ × 3″ × ¼″ └.
FG „ = − 11,595 „ } say 1/3″ × 3″ × ¼″ └.
FC „ = − 5,142 „ say 1/2½″ × 2½″ × ¼″ └.

Owing to the length of the ties (about 19 ft.), they should be suspended about the centre from the adjacent purlin.

The section for the struts is determined by the limit of $\frac{L}{R}$, i.e. $\frac{160}{1}$. If desired the ┐┌ struts may be dispensed with and the purlins at these points utilized in their stead, the purlins being designed to resist the combined compressive and bending actions.

The reacting force to that in member AF may be resolved into three components, i.e. one (R) acting laterally—across the gable—from eaves to eaves, one (R^1) acting longitudinally—along the eaves—from end to end, and the other vertically, the position coinciding with the centre of the corner stancheon. The last force will be amply provided for, and the effect may be ignored. Reaction R will equal stress in $AF \times \frac{AC}{AG} = 16{,}735 \times \frac{30}{40 \cdot 8} = 12{,}305$ lbs., and the nature of the stress in the resisting member will be compressive. This member will be composed of 2/3″×2″×¼″ ┐┌, and as the thrust in this member is taken up by the diagonal tie in the gable—between the corner and first intermediate stancheons—its length will equal the distance between these stancheons. If desired this strut may be dispensed with and the gable rail utilized, the latter being designed accordingly. Reaction R^1 will equal stress in $AF \times \frac{CG}{AG} = 16{,}735 \times \frac{24}{40 \cdot 8}$ $= 9844$ lbs., and the nature of the force will be compressive. This member will also be of 2/3″×2″×¼″ ┐┌; and the length will be that between the corner stancheon and the first side stancheon. This member may also be dispensed with by utilizing the eaves rail or purlin, or both, providing the stress is allowed for.

Gable Braces.—These braces extend diagonally across the first gable panel at each corner of the building, as shown on Plate 1/A. The functions are :—

(1) To transmit the end wind force from the aforementioned struts to the foundations.

(2) To prevent distortion in the gable due to the action of the roof wind force acting on the first 6′-0″ length of the building and to transmit such force to the foundations.

(3) and (4) As described for roof bracings.

The total force to be resisted is of a tensile nature, and the amount may be assumed with safety as due to (1), and will equal $12{,}305 \times \frac{20}{12}$ $= 20{,}508$ lbs., which will be resisted by a 3½″×½″ flat bar.

Side Braces.—These braces extend diagonally across the first side panel at each corner of the building, as shown on Plate 1/A. The functions are somewhat similar to those of the gable braces, and the section may be the same, i.e. 3½″×½″ flat bar.

Sag rods.—These are provided to the side and end rails and to the roof purlins, as before described, the length being equal to the distance between the adjacent rails or purlins. In the sheds under discussion the maximum tension is that in the top gable rod, i.e. about 851 lbs., and for practical reasons the rods will be ⅝″ diameter, screwed at each end and provided with nuts for securing to the rails and purlins.

Foundations.—The design of the foundations is governed by the nature of the ground and by the forces to be resisted. The method of designing foundations where proper fixity of base is required, as for Types 2 and 4, is very complex, and, if theory is to be strictly adhered to, is commercially impracticable.

Generally speaking, two methods may be followed, i.e. :—

(1) Where the resistance of the foundation is derived from a mass of concrete of such dimensions as to allow of the resultant force (lateral and vertical forces combined) intersecting the base of the concrete at a point away from the centre of about one-third to two-thirds the distance between the centre and the inner edge of the concrete mass, that is, the edge to which the resultant is pointing, the exact position being fixed by the allowable pressure.

(2) Where the resistance of the foundations is derived from a mass of concrete as described in (1), plus the resistance of two forces acting as a " couple," such forces being situated at the centre of action of the resistance offered to overturning by the earth, at the windward and leeward sides of the foundations.

The objection to the first method is that such method is commercially impracticable ; whilst the objection to the second method is that the earth will not comply with " the elastic law," and therefore the fixity will be imperfect.

In spite of the objection to method (2), this method is the one generally used in the trade, and has proved satisfactory, probably owing to the estimated allowance for wind pressure exceeding the actual amount, and also owing to the actual stresses exceeding the estimated stresses.

The choice between these Types as to whether to adopt Type 3 or Type 4, or a compromise (Type 4 foundations and a mean between Type 3 and Type 4 stancheons) is left to the designer, but if the earth available is capable of safely resisting the pressure adopted for Type 4, i.e. 3 tons, then the author—as the result of experience—advises as follows :—

(1) If the design is non-competitive, adopt Type 3 always in preference to Type 4.

(2) If the design is competitive, or Type 3 unsuitable, as, for instance, a machine shop with crane, adopt either Type 4 or a mean between Types 3 and 4, but preferably the latter.

For the purpose of this chapter the safe allowable pressure for the earth will be taken as 3 tons per square foot for Types 2 and 4, and 1 ton per square foot for Type 3.

Side Foundations, Type 2.—The forces to be resisted by the side foundations are :—

(1) The horizontal component of the roof wind force (HN).
(2) The vertical „ „ „ „ „ „ (VN).
(3) The side wind force (HP^S).
(4) The weight of the roof (W), and of the side (W^1).

The moment due to forces (1) and (3) is resisted by two forces, one force acting on the windward side and the other on the leeward side of the foundation. The bottom portion of the stancheon is embedded in concrete, and the latter acts as the medium in equalizing the effect of action and reaction. The lateral forces acting on the stancheon tend to overturn the concrete block, such tendency being resisted by the earth on the leeward and windward sides. The pressure on the leeward side varies in intensity from zero, at the bottom, to a maximum at the top of the foundation, and the centre of pressure will therefore be at a distance, down from the top, of one-third the depth of the concrete. The pressure on the windward side also varies in intensity, being zero at the top and a maximum at the bottom, the centre of pressure being situated at a distance, up from the bottom, of one-third the depth of the concrete.

Assuming the concrete block to be 4′-0″ deep by 2′-9″ square, then the centre of pressure on the leeward and windward sides will be 1′-4″ and 2′-8″ from the top respectively, and the earth pressure may be found as follows :—

Lateral forces $\frac{HN}{2}=1485$ lbs., and $\frac{HP^S}{2}=2400$ lbs.

Leverage of HN $=20'\text{-}0''+1'\text{-}4''=21'\text{-}4''$.

Leverage of HP^S $=10'\text{-}0''+1'\text{-}4''=11'\text{-}4''$.

Total overturning moment$=(1485\times21'\text{-}4'')+(2400\times11'\text{-}4'')$
$=58,880$ ft.-lbs.

The vertical forces act through the centre line of the stancheon, and have a leverage equal to, say, one-third the width of the concrete block. These forces counteract the overturning effect of the lateral forces by the extent shown hereafter :—

Force (2) = ·7 VN = 4541 lbs.
„ (4) = $W+W^1$ = 4368 „
Weight of concrete, say = 3400 „
12,309 „

Resisting moment$=12,309$ lbs.$\times 11''=11,283$ ft.-lbs.

The total overturning moment to be resisted by the earth will therefore be :—

$58,880-11,283=47,597$ ft.-lbs.

Windward reaction$=\frac{47,597}{1'\text{-}4''}=35,698$ lbs.

Leeward reaction$=35,698+1485+2400=39,583$ lbs.

Area of earth in contact with concrete = 4′-0″ × 2′-9″ = 11 sq. ft.

Mean pressure = 39,583 ÷ 11 = 3598 lbs.

Maximum pressure = 3598 lbs. × 2 = 7196 lbs., which is beyond safe limits.

Therefore increase the width to 3′-0″, which will reduce the maximum pressure from 7196 to 6596 lbs.

The effect of the vertical forces (2) and (4) is to put pressure on the earth in contact with the base of the concrete block, such pressure being unequally distributed over half the whole area, the amount being 12,309 lbs., and as the area of half the base is 4·125 sq. ft., the mean pressure will be 2984 lbs. per sq. ft., and maximum pressure 2984 × 2 = 5968 lbs., which is well within safe limits.

The size of block (4′-0″ deep × 2′-9″ × 3′-0″ wide) will therefore be suitable, and the stancheon will be embedded for a depth of 3′-0″ to 3′-3″.

Side Foundations, Type 3.—The base of the stancheon being assumed as pin-jointed, then the moment of the wind is resisted by the transverse frame; consequently the only forces acting on the foundations are the shearing force and the vertical force. For this type the safe earth pressure is assumed as 2240 lbs. per square foot, the design of shed being adopted for use on bad ground; the dimensions of the concrete block will be determined as follows :—

Shearing force = 4777 lbs.

Leeward area = 4777 ÷ 2240 = 2·1 sq. ft. (friction being ignored).

Vertical forces =

Weight of concrete block (2′-0″ × 2′-0″ × 1′-3″) =	560 lbs.
Vertical forces =	8087 „
Total	8647 „

Area of base = 8647 ÷ 2240 = 3·86 sq. ft.

The size of block assumed, i.e. 2′-0″ × 2′-0″ × 1′-3″, will therefore be ample. If the stancheon base rests *on* the concrete foundation the bolts forming the connection should be proportioned to resist the above shearing force.

Side Foundations, Type 4.—The procedure for this type is similar to that described for Type 2. The lateral forces are :—

(1) Lateral force at point of contraflexure in stancheon = 3427 lbs.

(3) „ „ on portion of stancheon below point of contraflexure = 1800 lbs.

Assuming the concrete block to be 3′-6″ deep × 2′-6″ square, the centre of pressure will be 1′-2″ and 2′-4″ down from the top, for the leeward and windward sides respectively; therefore the leverage for

Force (1) = 7′-6″ + 1′-2″ = 8′-8″, and for

„ (3) = 3′-9″ + 1′-2″ = 4′-11″, and the overturning moment will be (3427 × 8′-8″) + (1800 × 4′-11″) = 38,551 ft.-lbs.

The resistance offered by the vertical forces will be :—

Vertical forces	=	7947 lbs.
Weight of concrete	=	2450 ,,
Total		10,397 ,,

Resistance = 10,397 × 10″ = 8664 ft.-lbs.

Total moment to be resisted by earth = 38,551 − 8664 = 29,887 ft.-lbs.

Windward reaction = 29,887 ÷ 1′-2″ = 25,617 lbs.

Leeward ,, = 25,617 + 3427 + 1800 = 30,844 lbs.

Area of earth = 3′-6″ × 2′-6″ = 8·75 sq. ft.

Mean pressure = 30,844 ÷ 8·75 = 3525 lbs. per sq. ft.

Maximum pressure = 3525 × 2 = 7050 lbs. per sq. ft.

This pressure exceeds safe limits, therefore increases the width to 3′-0″.

The vertical force is 10,397 lbs., and the effective base area 3¾ sq. ft.; therefore the mean pressure per square foot = 2772 lbs., and the maximum pressure equals 2772 × 2 = 5544 lbs.

The earth pressures are within the safe limit, and the size of block adopted, i.e. 3′-6″ deep × 2′-6″ × 3′-0″ wide, is suitable. The stancheon should be embedded for a depth of about 2′-9″.

Gable Foundations.—The procedure for these foundations is similar to that for the side foundations for Type 3. The forces to be resisted are the same for all types, i.e. the vertical force due to dead weight and the shearing force due to wind. Taking the centre stancheon for the maximum loads, these will be :—

Vertical force, say, 5190 lbs.

Wind reaction ,, 5062 ,,

For an allowable pressure of 6720 lbs. (Types 2 and 4) the concrete block would do if 1′-0″ cube, but for practical reasons the block will require to be 2′-0″ × 1′-6″ × 1′-6″ deep, and this size will be suitable for the lower pressure of 2240 lbs. adopted for Type 3. The stancheon base should be embedded about 9″ to resist the shearing force.

The foundations are all shown on Plate 1/A.

Sheds of Multiple Bays.—The application of the theory of structures as before described for single bay sheds is somewhat similar for sheds having two or more bays. The cardinal point to be considered before the reactions can be determined is that of the distribution of the lateral forces. If these forces can be distributed over the entire width without subjecting the bottom chords of the roof principals to a compressive stress, then this distribution will be the most correct and most economical. If, however, the amount of side wind force to be transmitted through the principal of the windward bay, exceeds the tension in the centre tie, due to the thrusting effect of the vertical load on each half of the principal, then the bottom chord of the principal will be in compression, and as the chord will need to have proper side stiffeners (longitudinal ties) and be designed as a strut, this arrangement will be uneconomical. The alternative method will be to limit the side wind force transmitted through the principal to something less than the tensile force referred to above.

For practical purposes, and to facilitate the standardizing and repetition of the various parts, it is preferable to proportion the outer bays as if they were single bay sheds, and to adopt the same sections for the inner bays, and this procedure will not involve any great uneconomic use of material. If desired, as the tendency to distortion is less in the inner than in the outer bays, the stancheons for the inner bays may be decreased in strength, or placed further apart longitudinally.

Framed Sheds with Brick Panels.—Although the types chosen for illustration have the walls covered with corrugated sheeting, the stresses for a shed having brick panelling between the stancheons may be determined in a similar manner, with the proviso that as the brick wall will be considerably heavier than the sheet covering, the resistance to overturning may be utilized and the sections of the principals and stancheons decreased accordingly.

CHAPTER II.

THE STANDARDIZING OF SHED-MEMBERS AND FOUNDATIONS.

SHEETED SHEDS.

By adopting a constant outline for the principal members of the roof principals and transverse frames, by using a constant relative proportion for the main dimensions, and also a similar combination of data, stress coefficients may be obtained, which, when multiplied by certain necessary varying factors, will give the total stresses in the various members and thus greatly facilitate designing. Further, by the compiling and use of a standard table of sections for sheds of varying dimensions, the manufacture and erection will be considerably accelerated and cheapened.

The data adopted for use in the determination of the unit stresses are as follows :—

Weight of roof : 8 lbs. per sup. ft.
,, sides : 5 ,, ,,
Roof wind force : 32 lbs. per sq. ft.—horizontal.
Side ,, ,, 20 ,, ,, ,,
Angle of roof slope : 24½°.
Length of building considered : 1′-0″.
Span (transverse distance between side stancheons) : 12-′0″.
Height from ground to eaves : 4′-0″.
Unit length $\times\frac{\text{Span}}{12}$ = total length of member.

Force Constants for Reactions.

The constants previously referred to will similarly apply and may be found as follows :—

HP for roof : 32 lbs. per sq. ft. Resolving these, then
NP for roof = 32 × ·56 = 18 lbs. per sq. ft.
VN ,, ,, = 18 × ·91 = 16·38 lbs. per sq. ft.
HN ,, ,, = 18 × ·4146 = 7·46 ,, ,,
Length of roof slope relative to span = 6·6 ft.
Area of roof = 6·6 sq. ft.
Total VN = 16·38 × 6·6 = 108·11 lbs.
,, HN = 7·46 × 6·6 = 49·24 lbs.
,, NP = 18 × 6·6 = 118·8 lbs.

HP for side$=20$ lbs. per. sq. ft.
Height of side$=4'$-$0''$.
Area of side$=4$ sq. ft.
$\text{HP}^{\text{S}}=20\times 4=80$ lbs.
Weight of roof $(\text{W})=16\times 6{\cdot}6=105{\cdot}6$ lbs.
Weight of each side $(\text{W}^1)=5\times 4=20$ lbs.

CALCULATIONS FOR REACTIONS.

Type 2, Principals.

$$\text{Total upward reaction at WRP}=\frac{\text{W}}{2}+{\cdot}637\ \text{NP}=52{\cdot}8+75{\cdot}675$$
$$=128{\cdot}475\ \text{lbs.}$$

$$\text{Total upward reaction at LRP}=\frac{\text{W}}{2}+{\cdot}275\ \text{NP}=52{\cdot}8+32{\cdot}432$$
$$=85{\cdot}232\ \text{lbs.}$$

Total horizontal reaction at WRP$={\cdot}2073$ NP$=24{\cdot}627$ lbs.

„ „ „ „ LRP$={\cdot}2073$ NP$+{\cdot}25$ $\text{HP}^{\text{S}}=24{\cdot}627+20$
$=44{\cdot}627$ lbs.

Type 3, Transverse Frames.

$$\text{Total upward reaction at WRT}=\frac{\text{W}}{2}+\text{W}^1+\left[\frac{\text{C}-({\cdot}3\text{C}+{\cdot}4557\text{S}^2)}{\text{C}}\times\text{VN}\right]$$
$$-\frac{\text{HP}^{\text{S}}\times\text{L}^2}{\text{C}}=52{\cdot}8+20+59{\cdot}2-13{\cdot}33=118{\cdot}67\ \text{lbs.}$$

$$\text{Total upward reaction at LRT}=\frac{\text{W}}{2}+\text{W}^1+\left[\frac{({\cdot}3\text{C}+{\cdot}4557\text{S}^2)}{\text{C}}\times\text{VN}\right]$$
$$-\frac{\text{HP}^{\text{S}}\times\text{L}^2}{\text{C}}=52{\cdot}8+20+48{\cdot}8+13{\cdot}33=134{\cdot}93\ \text{lbs.}$$

$$\text{Total horizontal reaction at WRT}={\cdot}2073\ \text{NP}+\frac{\text{HP}^{\text{S}}-\text{RHP}}{2}+\text{RHP}$$
$$=24{\cdot}627+25+30=79{\cdot}627\ \text{lbs.}$$

$$\text{Total horizontal reaction at LRT}={\cdot}2073\ \text{NP}+\frac{\text{HP}^{\text{S}}-\text{RHP}}{2}$$
$$=24{\cdot}627+25=49{\cdot}627\ \text{lbs.}$$

Type 4, Transverse Frames.

$$\text{Total upward reaction at WRT}=\frac{\text{W}}{2}+\text{W}^1+\left[\frac{\text{C}-({\cdot}3\text{C}+{\cdot}4557\text{S}^2)}{\text{C}}\times\text{VN}\right]$$
$$-\frac{\text{HP}^{\text{S}}\times\text{L}^2}{\text{C}}=52{\cdot}8+20+65{\cdot}34-5{\cdot}2=132{\cdot}94\ \text{lbs.}$$

Total upward reaction at $\text{LRT}=\frac{W}{2}+W^1+\left[\frac{\cdot 3C+\cdot 4557S^2}{C}\times \text{VN}\right]$

$+\frac{\text{HP}^S\times L^2}{C}=52\cdot 8+20+42\cdot 65+5\cdot 2=120\cdot 65$ lbs.

Total horizontal reaction at $\text{WRT}=\cdot 2073\ \text{NP}+\frac{\text{HP}^S-\text{RHP}}{2}+\text{RHP}$

$=24\cdot 627+17\cdot 5+15=57\cdot 127$ lbs.

Total horizontal reaction at $\text{LRT}=\cdot 2073\ \text{NP}+\frac{\text{HP}^S-\text{RHP}}{2}$

$=24\cdot 627+17\cdot 5=42\cdot 127$ lbs.

These reactions will enable the force diagrams to be completed, from which the stress diagrams may be drawn in the usual manner, as shown on Plates 2/E, 2/F, and 2/G for Types 2, 3, and 4 respectively, and the amount and nature of the various unit stresses in the members determined. These unit stresses are tabulated on sheet 2/1 for spans ranging from 40 ft. to 70 ft., and on sheet 2/2 for spans from 70 ft. to 100 ft.

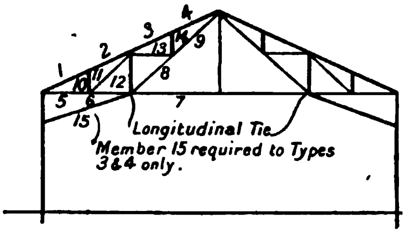
Since these unit stresses have been determined for sheds having the fixed ratio of span to height of 12 : 4, and for principals supporting a length of building equal to 1 foot, then, if the ratio adopted is adhered to, the total stress in any member may be found by the formula :—

$S=\text{US}\times\frac{C}{12}\times D$, where

S = total stress in member.
US = unit „ „ „
C = span in feet.
D = distance apart, in feet, of principals or transverse frames.

UNIT STRESSES IN SPANS 40 TO 70 FT.

SHEETED ROOFS.



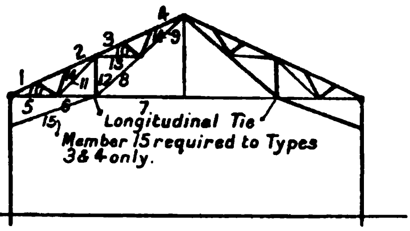
Member	Condition of Ends	Unit Length	* Unit stress in pounds		
			Type 2	Type 3	Type 4
1	B.E.F.	1·6484	+261	+324	+281
2	B.E.F.	1·6484	+274	+337	+294
3	B.E.F.	1·6484	+287	+350	+307
4	B.E.F.	1·6484	+300	+363	+320
5		1·5	−235	−235	−185
6		1·5	−184	−220	−170
7		6	−83	−70	−78
8	O.E.F.	2·0296	−137	+57 or −224	+20 or −172
9	O.E.F.	2·0296	−205	+39 or −292	+2 or −240
10	O.E.F.	·6836	+46	+46	+46
11		2·0296	−68	−68	−68
12	B.E.F.	1·3672	+92	+92	+92
13		1·5	−51	−51	−51
14	O.E.F.	·6836	+46	+46	+46
15	B.E.F.	3·1623		+212	+113

B.E.F. denotes both ends fixed.
O.E.F. denotes one end fixed, other end free.
Unit length multiplied by multiplier equals length of member.
* + denotes compression.
* — denotes tension.

SHEET 2/1

UNIT STRESSES IN SPANS 70 TO 100 FT.

SHEETED ROOFS.



Member	Condition of Ends	Unit Length	* Unit stress in pounds		
			Type 2	Type 3	Type 4
1	B.E.F.	1·099	+293	+340	+297
2	B.E.F.	1·099	+293	+356	+313
3	B.E.F.	1·099	+305	+368	+325
4	B.E.F.	1·099	+321	+384	+341
5		1·5	−250	−250	−195
6		1·5	−184	−220	−170
7		6	−83	−70	−78
8	O.E.F.	2·0296	−137	+57 or −224	+20 or −172
9	O.E.F.	2·0296	−226	+18 or −313	+2 or −261
10	O.E.F.	·6761	+31	+31	+31
11		2·0296	−89	−89	−89
12	B.E.F.	1·3672	+92	+92	+92
13		1·5	−66	−66	−66
14	O.E.F.	1·04	+46	+46	+46
15	B.E.F.	3·1623		+212	+113

B.E.F. denotes both ends fixed. SHEET 2/2

O.E.F. denotes one end fixed, other end free.

Unit length multiplied by multiplier equals length of member.

* + denotes compression.

* − denotes tension.

The proportion of the height to the span and the value of the expression $\frac{C}{12} \times D$ are given below for various spans :—

		$\frac{C}{12} \times D$ equals for	
Span	Height	12′-0″ centres	15′-0″ centres
40′-0″	13′-4″	40	50
50′-0″	16′-8″	50	62·5
60′-0″	20′-0″	60	75
70′-0″	23′-4″	70	87·5
80′-0″	26′-8″	80	100
90′-0″	30′-0″	90	112·5
100′-0″	33′-4″	100	125

To illustrate the utility of the above values :—

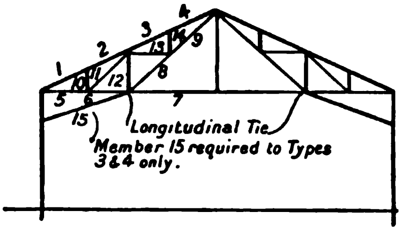
The total stress in any member for a shed 60′-0″ span × 20′-0″ high × 12′-0″ centres may be determined by multiplying the unit stress in that member by 60 ; if the centres are 15′-0″ then the multiplier will be 75.

By the use of the tables for unit stresses and multipliers, the total stresses in the various members for sheds of varying spans (but with height at the fixed ratio to the span) may be easily found. Sheets 2/3 to 2/8 give the tabulated stresses for spans varying from 40 to 70 ft., and sheets 2/9 to 2/14 for spans varying from 70 to 100 ft. It should be noted that the frame diagram for the latter range of spans is a modification of that used for finding the unit stresses; consequently, for certain members, the stresses will require to be separately computed, and although no diagram nor calculations are given, the unit stresses given on sheets 2/9 to 2/14, for these particular members, may be safely accepted as correct.

TOTAL STRESSES IN SPANS 40 TO 70 FT.

12′-0″ CENTRES—TYPE 2.

SHEETED ROOFS.



Member	Condition of Ends	Unit Length	* Total stress			
			40′-0″ Span	50′-0″ Span	60′-0″ Span	70′-0″ Span
			pounds	pounds	pounds	pounds
1	B.E.F.	1·6484	+10440	+13050	+15660	+18270
2	B.E.F.	1·6484	+10960	+13700	+16440	+19180
3	B.E.F.	1·6484	+11480	+14350	+17220	+20090
4	B.E.F.	1·6484	+12000	+15000	+18000	+21000
5		1·5	− 9400	−11750	−14100	−16450
6		1·5	− 7360	− 9200	−11040	−12880
7		6	− 3320	− 4150	− 4980	− 5810
8		2·0296	− 5480	− 6850	− 8220	− 9590
9		2·0296	− 8200	−10250	−12300	−14350
10	O.E.F.	·6836	+ 1840	+ 2300	+ 2760	+ 3220
11		2·0296	− 2720	− 3400	− 4080	− 4760
12	B.E.F.	1·3672	+ 3680	+ 4600	+ 5520	+ 6440
13		1·5	− 2040	− 2550	− 3060	− 3570
14	O.E.F.	·6836	+ 1840	+ 2300	+ 2760	+ 3220

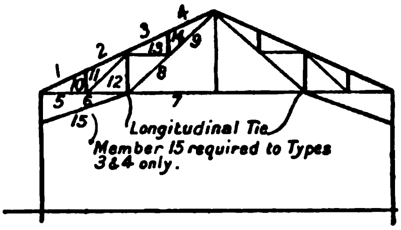
B.E.F. denotes both ends fixed.
O.E.F. denotes one end fixed, other end free.
Unit length multiplied by multiplier equals length of member.
* + denotes compression.
* − denotes tension.

Sheet 2/3

TOTAL STRESSES IN SPANS 40 TO 70 FT.

15′-0″ CENTRES—TYPE 2.

SHEETED ROOFS.



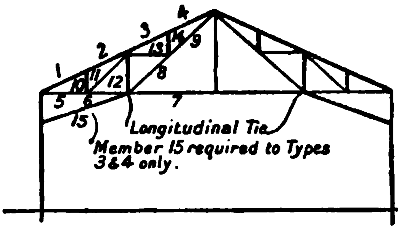
Member	Condition of Ends	Unit Length	* Total stress			
			40′-0″ Span	50′-0″ Span	60′-0″ Span	70′-0″ Span
			pounds	pounds	pounds	pounds
1	B.E.F.	1·6484	+13050	+16312	+19575	+22837
2	B.E.F.	1·6484	+13700	+17125	+20550	+23975
3	B.E.F.	1·6484	+14350	+17937	+21525	+25113
4	B.E.F.	1·6484	+15000	+18750	+22500	+26250
5		1·5	−11750	−14687	−17625	−20562
6		1·5	− 9200	−11500	−13800	−16100
7		6	− 4150	− 5187	− 6225	− 7262
8		2·0296	− 6850	− 8562	−10275	−11987
9		2·0296	−10250	−12812	−15375	−17937
10	O.E.F.	·6836	+ 2300	+ 2875	+ 3450	+ 4025
11		2·0296	− 3400	− 4250	− 5100	− 5950
12	B.E.F.	1·3672	+ 4600	+ 5750	+ 6900	+ 8050
13		1·5	− 2550	− 3187	− 3825	− 4462
14	O.E.F.	·6836	+ 2300	+ 2875	+ 3450	+ 4025

B.E.F. denotes both ends fixed. SHEET 2/4
O.E.F. denotes one end fixed, other end free.
Unit length multiplied by multiplier equals length of member.
* + denotes compression.
* − denotes tension.

TOTAL STRESSES IN SPANS 40 TO 70 FT.

12'-0" CENTRES—TYPE 3.

SHEETED ROOFS.



Member	Condition of Ends	Unit Length	* Total stress			
			40'-0" Span	50'-0" Span	60'-0" Span	70'-0" Span
			pounds	pounds	pounds	pounds
1	B.E.F.	1·6484	+12960	+16200	+19440	+22680
2	B.E.F.	1·6484	+13480	+16850	+20220	+23590
3	B.E.F.	1·6484	+14000	+17500	+21000	+24500
4	B.E.F.	1·6484	+14520	+18150	+21780	+25410
5		1·5	− 9400	−11750	−14100	−16450
6		1·5	− 8800	−11000	−13200	−15400
7		6	− 2800	− 3500	− 4200	− 4900
8	O.E.F.	2·0296	+ 2280 or − 8960	+ 2850 or −11200	+ 3420 or −13440	+ 3990 or −15680
9	O.E.F.	2·0296	+ 1560 or −11680	+ 1950 or −14600	+ 2340 or −17520	+ 2730 or −20440
10	O.E.F.	·6836	+ 1840	+ 2300	+ 2760	+ 3220
11		2·0296	− 2720	− 3400	− 4080	− 4760
12	B.E.F.	1·3672	+ 3680	+ 4600	+ 5520	+ 6440
13		1·5	− 2040	− 2550	− 3060	− 3570
14	O.E.F.	·6836	+ 1840	+ 2300	+ 2760	+ 3220
15	B.E.F.	3·1623	+ 8480	+10600	+12720	+14840

B.E.F. denotes both ends fixed. SHEET 2/5

O.E.F. denotes one end fixed, other end free.

Unit length multiplied by multiplier equals length of member.

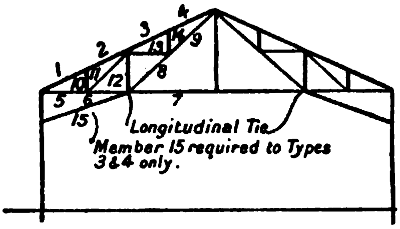
* + denotes compression.

* − denotes tension.

TOTAL STRESSES IN SPANS 40 TO 70 FT.

15′-0″ CENTRES—TYPE 3.

SHEETED ROOFS.



Member	Condition of Ends	Unit Length	* Total stress			
			40′-0″ Span	50′-0″ Span	60′-0″ Span	70′-0″ Span
			pounds	pounds	pounds	pounds
1	B.E.F.	1·6484	+16200	+20250	+24300	+28350
2	B.E.F.	1·6484	+16850	+21062	+25275	+29487
3	B.E.F.	1·6484	+17500	+21875	+26250	+30625
4	B.E.F.	1·6484	+18150	+22687	+27225	+31762
5		1·5	−11750	−14687	−17625	−20562
6		1·5	−11000	−13750	−16500	−19250
7		6	− 3500	− 4375	− 5250	− 6125
8	O.E.F.	2·0296	+ 2850 or −11200	+ 3562 or −14000	+ 4275 or −16800	+ 4987 or −19600
9	O.E.F.	2·0296	+ 1950 or −14600	+ 2437 or −18250	+ 2925 or −21900	+ 3412 or −25550
10	O.E.F.	·6836	+ 2300	+ 2875	+ 3450	+ 4025
11		2·0296	− 3400	− 4250	− 5100	− 5950
12	B.E.F.	1·3672	+ 4600	+ 5750	+ 6900	+ 8050
13		1·5	− 2550	− 3187	− 3825	− 4462
14	O.E.F.	·6836	+ 2300	+ 2875	+ 3450	+ 4025
15	B.E.F.	3·1623	+10600	+13250	+15900	+18550

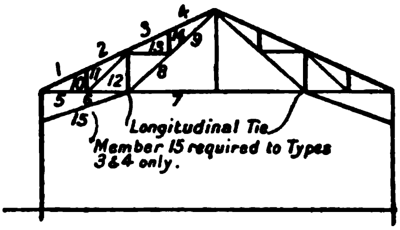
B.E.F. denotes both ends fixed.
O.E.F. denotes one end fixed, other end free.
Unit length multiplied by multiplier equals length of member.
* + denotes compression.
* − denotes tension.

SHEET 2/6

TOTAL STRESSES IN SPANS 40 TO 70 FT.

12′-0″ CENTRES—TYPE 4.

SHEETED ROOFS.



Member	Condition of Ends	Unit Length	* Total stress			
			40′-0″ Span	50′-0″ Span	60′-0″ Span	70′-0″ Span
			pounds	pounds	pounds	pounds
1	B.E.F.	1·6484	+11240	+14050	+16860	+19670
2	B.E.F.	1·6484	+11760	+14700	+17640	+20580
3	B.E.F.	1·6484	+12280	+15350	+18420	+21490
4	B.E.F.	1·6484	+12800	+16000	+19200	+22400
5		1·5	− 7400	− 9250	−11100	−12950
6		1·5	− 6800	− 8500	−10200	−11900
7		6	− 3120	− 3900	− 4680	− 5460
8	O.E.F.	2·0296	+ 800 or − 6880	+ 1000 or − 8600	+ 1200 or −10320	+ 1400 or −12040
9	O.E.F.	2·0296	+ 80 or − 9600	+ 100 or −12000	+ 120 or −14400	+ 140 or −16800
10	O.E.F.	·6836	+ 1840	+ 2300	+ 2760	+ 3220
11		2·0296	− 2720	− 3400	− 4080	− 4760
12	B.E.F.	1·3672	+ 3680	+ 4600	+ 5520	+ 6440
13		1·5	− 2040	− 2550	− 3060	− 3570
14	O.E.F.	·6836	+ 1840	+ 2300	+ 2760	+ 3220
15	B.E.F.	3·1623	+ 4520	+ 5650	+ 6780	+ 7910

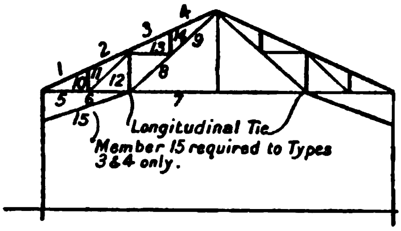
B.E.F. denotes both ends fixed.
O.E.F. denotes one end fixed, other end free.
Unit length multiplied by multiplier equals length of member.
* + denotes compression.
* − denotes tension.

Sheet 2/7

TOTAL STRESSES IN SPANS 40 TO 70 FT.

15′-0″ CENTRES—TYPE 4.

SHEETED ROOFS.



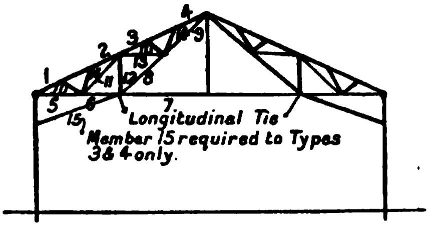
Member	Condition of Ends	Unit Length	* Total stress			
			40′-0″ Span	50′-0″ Span	60′-0″ Span	70′-0″ Span
			pounds	pounds	pounds	pounds
1	B.E.F.	1·6484	+14050	+17562	+21075	+24587
2	B.E.F.	1·6484	+14700	+18375	+22050	+25475
3	B.E.F.	1·6484	+15350	+19187	+23025	+26862
4	B.E.F.	1·6484	+16000	+20000	+24000	+28000
5		1·5	− 9250	−11562	−13875	−16187
6		1·5	− 8500	−10625	−12750	−14875
7		6	− 3900	− 4875	− 5850	− 6825
8	O.E.F.	2·0296	+ 1000 or − 8600	+ 1250 or −10750	+ 1500 or −12900	+ 1750 or −15050
9	O.E.F.	2·0296	+ 100 or −12000	+ 125 or −15000	+ 150 or −18000	+ 175 or −21000
10	O.E.F.	·6836	+ 2300	+ 2875	+ 3450	+ 4025
11		2·0296	− 3400	− 4250	− 5100	− 5950
12	B.E.F.	1·3672	+ 4600	+ 5750	+ 6900	+ 8050
13		1·5	− 2550	− 3187	− 3825	− 4462
14	O.E.F.	·6836	+ 2300	+ 2875	+ 3450	+ 4025
15	B.E.F.	3·1623	+ 5650	+ 7062	+ 8475	+ 9887

B.E.F. denotes both ends fixed. Sheet 2/8
O.E.F. denotes one end fixed, other end free.
Unit length multiplied by multiplier equals length of member.
* + denotes compression.
* − denotes tension.

TOTAL STRESSES IN SPANS 70 TO 100 FT.

12′-0″ CENTRES—TYPE 2.

SHEETED ROOFS.



Member	Condition of Ends	Unit Length	* Total stress			
			70′-0″ Span	80′-0″ Span	90′-0″ Span	100′-0″ Span
			pounds	pounds	pounds	pounds
1	B.E.F.	1·099	+19390	+22160	+24930	+27700
2	B.E.F.	1·099	+20510	+23440	+26370	+29300
3	B.E.F.	1·099	+21350	+24400	+27450	+30500
4	B.E.F.	1·099	+22470	+25680	+28890	+32100
5		1·5	−17500	−20000	−22500	−25000
6		1·5	−12880	−14720	−16560	−18400
7		6	− 5810	− 6640	− 7470	− 8300
8		2·0296	− 9590	−10960	−12330	−13700
9		2·0296	−15820	−18080	−20340	−22600
10	O.E.F.	·6761	+ 2170	+ 2480	+ 2790	+ 3100
11		2·0296	− 6230	− 7120	− 8010	− 8900
12	B.E.F.	1·3672	+ 6440	+ 7360	+ 8280	+ 9200
13		1·5	− 4620	− 5280	− 5940	− 6600
14	O.E.F.	1·04	+ 3220	+ 3680	+ 4140	+ 4600

B.E.F. denotes both ends fixed.

SHEET 2/9

O.E.F. denotes one end fixed, other end free.

Unit length multiplied by multiplier equals length of member.

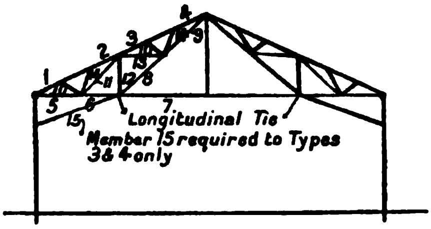
* + denotes compression.

* − denotes tension.

TOTAL STRESSES IN SPANS 70 TO 100 FT.

15′-0″ CENTRES—TYPE 2.

SHEETED ROOFS.



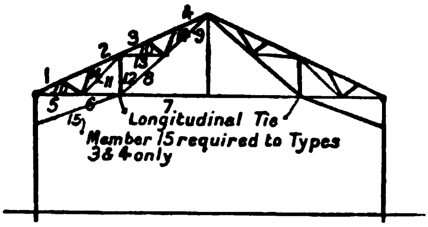
Member	Condition of Ends	Unit Length	* Total stress			
			70′-0″ Span	80′-0″ Span	90′-0″ Span	100′-0″ Span
			pounds	pounds	pounds	pounds
1	B.E.F.	1·099	+24238	+27700	+31162	+34625
2	B.E.F.	1·099	+25638	+29300	+32962	+36625
3	B.E.F.	1·099	+26688	+30500	+34312	+38125
4	B.E.F.	1·099	+28088	+32100	+36112	+40125
5		1·5	−21875	−25000	−28125	−31250
6		1·5	−16100	−18400	−20700	−23000
7		6	− 7262	− 8300	− 9338	−10375
8		2·0296	−11988	−13700	−15412	−17125
9		2·0296	−19775	−22600	−25425	−28250
10	O.E.F.	·6761	+ 2712	+ 3100	+ 3488	+ 3875
11		2·0296	− 7788	− 8900	−10012	−11125
12	B.E.F.	1·3672	+ 8050	+ 9200	+10350	+11500
13		1·5	− 5775	− 6600	− 7425	− 8250
14	O.E.F.	1·04	+ 4025	+ 4600	+ 5175	+ 5750

B.E.F. denotes both ends fixed. SHEET 2/10
O.E.F. denotes one end fixed, other end free.
Unit length multiplied by multiplier equals length of member.
* + denotes compression.
* − denotes tension.

TOTAL STRESSES IN SPANS 70 TO 100 FT.

12′-0″ CENTRES—TYPE 3.

SHEETED ROOFS.



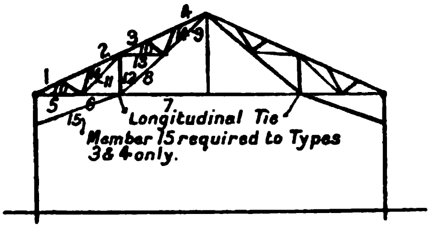
Member	Condition of Ends	Unit Length	* Total stress			
			70′-0″ Span	80′-0″ Span	90′-0″ Span	100′-0″ Span
			pounds	pounds	pounds	pounds
1	B.E.F.	1·099	+23800	+27200	+30600	+34000
2	B.E.F.	1·099	+24920	+28480	+32040	+35600
3	B.E.F.	1·099	+25760	+29440	+33120	+36800
4	B.E.F.	1·099	+26880	+30720	+34560	+38400
5		1·5	−17500	−20000	−22500	−25000
6		1·5	−15400	−17600	−19800	−22000
7		6	− 4900	− 5600	− 6300	− 7000
8	O.E.F.	2·0296	+ 3990 or −15680	+ 4560 or −17920	+ 5130 or −20160	+ 5700 or −22400
9	O.E.F.	2·0296	+ 1260 or −21910	+ 1440 or −25040	+ 1620 or −28170	+ 1800 or −31300
10	O.E.F.	·6761	+ 2170	+ 2480	+ 2790	+ 3100
11		2·0296	− 6230	− 7120	− 8010	− 8900
12	B.E.F.	1·3672	+ 6440	+ 7360	+ 8280	+ 9200
13		1·5	− 4620	− 5280	− 5940	− 6600
14	O.E.F.	1·04	+ 3220	+ 3680	+ 4140	+ 4600
15	B.E.F.	3·1623	+14840	+16960	+19080	+21200

B.E.F. denotes both ends fixed.
O.E.F. denotes one end fixed, other end free.
Unit length multiplied by multiplier equals length of member.
* + denotes compression.
* − denotes tension.

TOTAL STRESSES IN SPANS 70 TO 100 FT.

15′-0″ CENTRES—TYPE 3.

SHEETED ROOFS.



Member	Condition of Ends	Unit Length	* Total stress			
			70′-0″ Span	80′-0″ Span	90′-0″ Span	100′-0″ Span
			pounds	pounds	pounds	pounds
1	B.E.F.	1·099	+29750	+34000	+38250	+42500
2	B.E.F.	1·099	+31150	+35600	+40050	+44500
3	B.E.F.	1·099	+32200	+36800	+41400	+46000
4	B.E.F.	1·099	+33600	+38400	+43200	+48000
5		1·5	−21875	−25000	−28125	−31250
6		1·5	−19250	−22000	−24750	−27500
7		6	− 6125	− 7000	− 7875	− 8750
8	O.E.F.	2·0296	+ 4988 or −19600	+ 5700 or −22400	+ 6412 or −25200	+ 7125 or −28000
9	O.E.F.	2·0296	+ 1575 or −27388	+ 1800 or −31300	+ 2025 or −35212	+ 2250 or −39125
10	O.E.F.	·6761	+ 2712	+ 3100	+ 3488	+ 3875
11		2·0296	− 7788	− 8900	−10012	−11125
12	B.E.F.	1·3672	+ 8050	+ 9200	+10350	+11500
13		1·5	− 5775	− 6600	− 7425	− 8250
14	O.E.F.	1·04	+ 4025	+ 4600	+ 5175	+ 5750
15	B.E.F.	3·1623	+18550	+21200	+23850	+26500

B.E.F. denotes both ends fixed.

SHEET 2/12

O.E.F. denotes one end fixed, other end free.

Unit length multiplied by multiplier equals length of member.

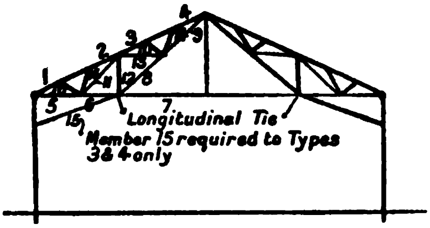
* + denotes compression.

* − denotes tension.

TOTAL STRESSES IN SPANS 70 TO 100 FT.

12′-0″ CENTRES—TYPE 4.

SHEETED ROOFS.



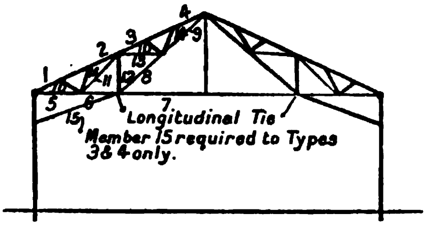
Member	Condition of Ends	Unit Length	* Total stress			
			70′-0″ Span	80′-0″ Span	90′-0″ Span	100′-0″ Span
			pounds	pounds	pounds	pounds
1	B.E.F.	1·099	+20790	+23760	+26730	+29700
2	B.E.F.	1·099	+21910	+25040	+28170	+31300
3	B.E.F.	1·099	+22750	+26000	+29250	+32500
4	B.E.F.	1·099	+23870	+27280	+30690	+34100
5		1·5	−13650	−15600	−17550	−19500
6		1·5	−11900	−13600	−15300	−17000
7		6	− 5460	− 6240	− 7020	− 7800
8	O.E.F.	2·0296	+ 1400 or −12040	+ 1600 or −13760	+ 1800 or −15480	+ 2000 or −17200
9	O.E.F.	2·0296	+ 140 or −18270	+ 160 or −20880	+ 180 or −23490	+ 200 or −26100
10	O.E.F.	·6761	+ 2170	+ 2480	+ 2790	+ 3100
11		2·0296	− 6230	− 7120	− 8010	− 8900
12	B.E.F.	1·3672	+ 6440	+ 7360	+ 8280	+ 9200
13		1·5	− 4620	− 5280	− 5940	− 6600
14	O.E.F.	1·04	+ 3220	+ 3680	+ 4140	+ 4600
15	B.E.F.	3·1623	+ 7910	+ 9040	+10170	+11300

B.E.F. denotes both ends fixed. SHEET 2/13
O.E.F. denotes one end fixed, other end free.
Unit length multiplied by multiplier equals length of member.
* + denotes compression.
* − denotes tension.

TOTAL STRESSES IN SPANS 70 TO 100 FT.

15′-0″ CENTRES—TYPE 4.

SHEETED ROOFS.



Member	Condition of Ends	Unit Length	70′-0″ Span	80′-0″ Span	90′-0″ Span	100′-0″ Span
			pounds	pounds	pounds	pounds
1	B.E.F.	1·099	+25988	+29700	+33412	+37125
2	B.E.F.	1·099	+27388	+31300	+35212	+39125
3	B.E.F.	1·099	+28438	+32500	+36562	+40625
4	B.E.F	1·099	+29838	+34100	+38362	+42625
5		1·5	−17063	−19500	−21937	−24375
6		1·5	−14875	−17000	−19125	−21250
7		6	− 6825	− 7800	− 8775	− 9750
8	O.E.F.	2·0296	+ 1750 or −15050	+ 2000 or −17200	+ 2250 or −19350	+ 2500 or −21500
9	O.E.F.	2·0296	+ 175 or −22838	+ 200 or −26100	+ 225 or −29362	+ 250 or −32625
10	O.E.F.	·6761	+ 2712	+ 3100	+ 3488	+ 3875
11		2·0296	− 7788	− 8900	−10012	−11125
12	B.E.F.	1·3672	+ 8050	+ 9200	+10350	+11500
13		1·5	− 5775	− 6600	− 7425	− 8250
14	O.E.F.	1·04	+ 4025	+ 4600	+ 5175	+ 5750
15	B.E.F.	3·1623	+ 9888	+11300	+12712	+14125

B.E.F. denotes both ends fixed.

SHEET 2/14

O.E.F. denotes one end fixed, other end free.
Unit length multiplied by multiplier equals length of member.
+ denotes compression.
− denotes tension.

STANDARD SECTIONS FOR PRINCIPALS AND TRANSVERSE FRAMES.

The sections recommended for the members of the principals and for the stays are given on sheets 2/15 to 2/18, and have been calculated to resist the tabulated total stresses, the stress per square inch being limited to :—

Allowable tensile stress : 7½ tons per square inch of net section.

Allowable compressive stress : 6½ tons per square inch of gross section reduced by the "Claxton Fidler" formula for struts to the allowable value for $\frac{L}{R}$, the latter being generally limited to the ratio $\frac{160}{1}$.

In calculating the sections of the rafters, the purlins have been assumed as situated directly over a strut. If this position is not adhered to, then flexural stress will be set up in addition to the direct compressive stress, and, as this condition frequently occurs in practice, alternative sections are given (in brackets), such sections being determined on the assumption that a purlin may be situated at the centre of the span between two adjacent struts.

To increase the utility of the standard stresses and sections, it is obvious that they should be adaptable to heights of sheds other than those given by the fixed ratio of height to span, i.e. 4 : 12, and this elasticity may to some extent be obtained as follows :—

Type 2.—The effect on the principals of increasing the height is to reduce the stresses in certain members, the amount varying in accordance with the amount of wind force transmitted to the leeward stancheon—which amount must not exceed the tensile stress in the centre tie and thus eliminate the possibility of compressive stress in this member—therefore the tabulated stresses and sections may, without uneconomic use of material, be assumed as unchanged.

If the height is decreased, then the reduction in the amount of wind force transmitted to the leeward stancheon should be added to the stresses in the bottom chord and the sections of the latter increased accordingly.

Types 3 and 4.—The transverse frames for similar spans of Types 3 and 4 vary only in relation to the distance (H^1) between the frame reaction point (R^T) and the principal reaction point (R^P)—the unit distance of the point R^P from the stay connection remains the same for each type. This unit distance H^1 is 4 for Type 3 and 2·5 for Type 4, therefore :—

Type 3.—For H^1 of unit distance of from 4 to 2·5, use stresses and sections as given for Type 3.

For H^1 equal to unit distance of 2·5 or less, use stresses and sections as for Type 4.

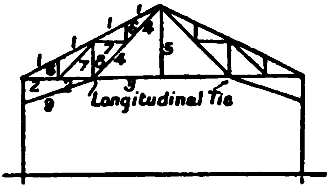
Type 4.—For H^1 equal to unit distance of 2·5 or less, use stresses and sections as for Type 4.

For H^1 equal to unit distance of 2·5 to 4, use stresses and sections as given for Type 3.

STANDARD SECTIONS FOR SPANS 40 TO 70 FT.

TYPE 2.

SHEETED ROOFS.



Member	40′-0″ Span	50′-0″ Span	60′-0″ Span	70′-0″ Span	Centres
1	2/2½″ × 2″ × ¼″ ┘└ (2/3″ × 2½″ × ¼″ ┘└)	2/3″ × 2″ × ¼″ ┘└ (2/3″ × 2½″ × $\frac{5}{16}$″ ┘└)	2/3″ × 2½″ × ¼″ ┘└ (2/3½″ × 2½″ × $\frac{5}{16}$″ ┘└)	2/3″ × 2½″ × $\frac{5}{16}$″ ┘└ (2/3½″ × 3″ × $\frac{5}{16}$″ ┘└)	12′-0″ Centres.
2	1/3″ × 2″ × ¼″ └	1/3″ × 3″ × ¼″ └	2/2½″ × 2″ × ¼″ ┘└	2/2½″ × 2″ × ¼″ ┘└	
3	1/2½″ × 2″ × ¼″ └	1/2½″ × 2″ × ¼″ └	2/2½″ × 2″ × ¼″ ┘└	2/2½″ × 2″ × ¼″ ┘└	
4	1/2½″ × 2″ × ¼″ └	1/2½″ × 2″ × ¼″ └	1/3″ × 2″ × ¼″ └	1/3″ × 2″ × ¼″ └	
5	1/2″ × ¼″ Flat	1/2″ × ¼″ Flat	1/2½″ × ¼″ Flat	1/2½″ × ¼″ Flat	
6	1/2″ × 2″ × ¼″ └	1/2″ × 2″ × ¼″ └	1/2½″ × 2½″ × ¼″ └	1/2½″ × 2½″ × ¼″ └	
7	1/2″ × 2″ × ¼″ └	1/2″ × 2″ × ¼″ └	1/2″ × 2″ × ¼″ └	1/2″ × 2″ × ¼″ └	
8	1/2½″ × 2½″ × ¼″ └	1/2½″ × 2½″ × ¼″ └	1/3″ × 3″ × ¼″ └	1/3″ × 3″ × $\frac{5}{16}$″ └	
1	2/2½″ × 2″ × ¼″ ┘└ (2/3″ × 2½″ × ¼″ ┘└)	2/3″ × 2″ × ¼″ ┘└ (2/3″ × 2½″ × $\frac{5}{16}$″ ┘└)	2/3″ × 2½″ × $\frac{5}{16}$″ ┘└ (2/3½″ × 3″ × $\frac{5}{16}$″ ┘└)	2/3″ × 2½″ × $\frac{5}{16}$″ ┘└ (2/4″ × 3″ × $\frac{5}{16}$″ ┘└)	15′-0″ Centres.
2	1/3″ × 2″ × ¼″ └	1/3″ × 3″ × ¼″ └	2/2½″ × 2″ × ¼″ ┘└	2/2½″ × 2″ × ¼″ ┘└	
3	1/2½″ × 2″ × ¼″ └	1/3″ × 2″ × ¼″ └	2/2½″ × 2″ × ¼″ ┘└	2/2½″ × 2″ × ¼″ ┘└	
4	1/2½″ × 2″ × ¼″ └	1/3″ × 2″ × ¼″ └	1/3½″ × 2″ × ¼″ └	1/3½″ × 2½″ × ¼″ └	
5	1/2″ × ¼″ Flat	1/2″ × ¼″ Flat	1/2½″ × ¼″ Flat	1/2½″ × ¼″ Flat	
6	1/2″ × 2″ × ¼″ └	1/2″ × 2″ × ¼″ └	1/2½″ × 2½″ × ¼″ └	1/2½″ × 2½″ × ¼″ └	
7	1/2″ × 2″ × ¼″ └	1/2″ × 2″ × ¼″ └	1/2″ × 2″ × ¼″ └	1/2″ × 2″ × ¼″ └	
8	1/2½″ × 2½″ × ¼″ └	1/2½″ × 2½″ × ¼″ └	1/3″ × 3″ × ¼″ └	1/3″ × 3″ × $\frac{5}{16}$″ └	

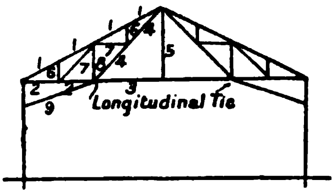
Sheet 2/15

Sections in brackets for Member " 1 " allow for flexural stress due to purlin in addition to "principal" stress.

STANDARD SECTIONS FOR SPANS 40 TO 70 FT.

TYPES 3 AND 4.

SHEETED ROOFS.



Member	40'-0" Span	50'-0" Span	60'-0" Span	70'-0" Span	Centres
1	2/2½"×2"×¼" ┘└ (2/3"×2½"×5/16" ┘└)	2/3" ×2" ×¼" ┘└ (2/3½"×2½"×5/16" ┘└)	2/3" ×2½"×¼" ┘└ (2/3½"×3" ×5/16" ┘└)	2/3½"×2½"×5/16" ┘└ (2/4" ×3" ×5/16" ┘└)	12'-0" Centres.
2	1/3" ×2" ×¼" └	1/3" ×3" ×¼" └	2/2½"×2" ×¼" ┘└	2/2½"×2" ×¼" ┘└	
3	1/3" ×2" ×¼" └	1/3" ×3" ×¼" └	2/2½"×2" ×¼" ┘└	2/2½"×2" ×¼" ┘└	
4	1/3" ×3" ×¼" └	2/2½"×2" ×¼" ┘└	2/2½"×2" ×¼" ┘└	2/3" ×2" ×¼" ┘└	
5	1/2" ×¼" Flat	1/2" ×¼" Flat	1/2½"×¼" Flat	1/2½"×¼" Flat	
6	1/2" ×2" ×¼" └	1/2" ×2" ×¼" └	1/2½"×2½"×¼" └	1/2½"×2½"×¼" └	
7	1/2" ×2" ×¼" └	1/2" ×2" ×¼" └	1/2" ×2" ×¼" └	1/2" ×2" ×¼" └	
8	1/2½"×2½"×¼" └	1/2½"×2½"×¼" └	1/3" ×3" ×¼" └	1/3" ×3" ×5/16" └	
9	2/2½"×2" ×¼" ┘└	2/3" ×2" ×¼" ┘└	2/3" ×3" ×¼" ┘┌	2/3" ×3" ×¼" ┘┌	
1	2/2½"×2" ×¼" ┘└ (2/3"×2½"×5/16" ┘└)	2/3" ×2" ×¼" ┘└ (2/3½"×2½"×5/16" ┘└)	2/3" ×2½"×5/16" ┘└ (2/3½"×3" ×5/16" ┘└)	2/3½"×3" ×5/16" ┘└ (2/4" ×3" ×5/16" ┘└)	15'-0" Centres.
2	1/3" ×2" ×¼" └	1/3" ×3" ×¼" └	2/2½"×2" ×¼" ┘└	2/2½"×2" ×¼" ┘└	
3	1/3" ×2" ×¼" └	1/3" ×3" ×¼" └	2/2½"×2" ×¼" ┘└	2/2½"×2" ×¼" ┘└	
4	1/3" ×3" ×¼" └	2/2½"×2" ×¼" ┘└	2/2½"×2" ×¼" ┘└	2/3" ×2" ×¼" ┘└	
5	1/2" ×¼" Flat	1/2" ×¼" Flat	1/2½"×¼" Flat	1/2½"×¼" Flat	
6	1/2" ×2" ×¼" └	1/2" ×2" ×¼" └	1/2½"×2½"×¼" └	1/2½"×2½"×¼" └	
7	1/2" ×2" ×¼" └	1/2" ×2" ×¼" └	1/2" ×2" ×¼" └	1/2" ×2" ×¼" └	
8	1/2½"×2½"×¼" └	1/3" ×3" ×¼" └	1/3" ×3" ×5/16" └	1/3" ×3" ×5/16" └	
9	2/3" ×2" ×¼" ┘└	2/3" ×2½"×¼" ┘┌	2/3" ×3" ×¼" ┘┌	2/3" ×3" ×5/16" ┘┌	

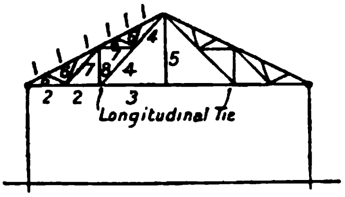
SHEET 2/16

Sections in brackets for Member "1" allow for flexural stress due to purlin in addition to "principal" stress.

STANDARD SECTIONS FOR SPANS 70 TO 100 FT.

TYPE 2.

SHEETED ROOFS.



Member	70′-0″ Span	80′-0″ Span	90′-0″ Span	100′-0″ Span	Centres
1	2/3″ ×2½″×¼″ ⅃L (2/3½″×2½″×5⁄16″ ⅃L)	2/3½″×2½″×¼″ ⅃L (2/3½″×2½″×⅜″ ⅃L)	2/3½″×2½″×5⁄16″ ⅃L (2/4″ ×3″ ×5⁄16″ ⅃L)	(2/3½″×2½″×5⁄16″ ⅃L) (2/4″ ×3″ ×5⁄16″ ⅃L)	12′-0″ Centres.
2	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	
3	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	
4	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	
5	1/2½″×2″ ×¼″ L	1/2½″×2″ ×¼″ L	1/2½″×2″ ×¼″ L	1/2½″×2″ ×¼″ L	
6	1/3″ ×3″ ×¼″ L	1/3″ ×3″ ×5⁄16″ L	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	
7	1/2½″×2″ ×¼″ L	1/2½″×2″ ×¼″ L	1/2½″×2″ ×¼″ L	1/2½″×2″ ×¼″ L	
8	1/3″ ×3″ ×¼″ L	1/3½″×3½″×5⁄16″ L	2/3″ ×2″ ×¼″ ⅃L	2/3″ ×2″ ×¼″ ⅃L	
1	2/3″ ×2½″×5⁄16″ ⅃L (2/3½″×3″×5⁄16″ ⅃L)	2/3½″×2½″×5⁄16″ ⅃L (2/4″ ×3″ ×5⁄16″ ⅃L)	2/3½″×2½″×⅜″ ⅃L (2/4″ ×3″ ×⅜″ ⅃L)	2/3½″×2½″×⅜″ ⅃L (2/4″ ×3″ ×⅜″ ⅃L)	15′-0″ Centres.
2	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	2/3″ ×2″ ×¼″ ⅃L	2/3″ ×2″ ×¼″ ⅃L	
3	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	
4	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	2/3″ ×2″ ×¼″ ⅃L	
5	1/2½″×2″ ×¼″ ⅃L	1/2½″×2″ ×¼″ L	1/2½″×2″ ×¼″ L	1/2½″×2″ ×¼″ L	
6	1/3″ ×3″ ×¼″ ⅃L	1/3″ ×3″ ×5⁄16″ L	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	
7	1/2½″×2″ ×¼″ ⅃L	1/2½″×2″ ×¼″ L	1/2½″×2″ ×¼″ L	1/2½″×2″ ×¼″ L	
8	1/3″ ×3″ ×5⁄16″ ⅃L	1/3½″×3½″×5⁄16″ L	2/3″ ×2″ ×¼″ ⅃L	2/3″ ×2½″×¼″ ⅃L	

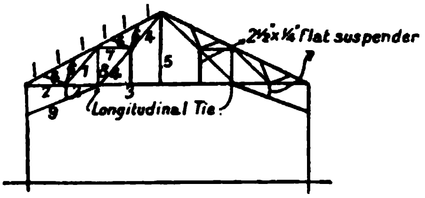
Sheet 2/17

Sections in brackets for Member " 1 " allow for flexural stress due to purlin in addition to " principal " stress.

STANDARD SECTIONS FOR SPANS 70 TO 100 FT.

TYPES 3 AND 4.

SHEETED ROOFS.



Member	70′-0″ Span	80′-0″ Span	90′-0″ Span	100′-0″ Span	Centres
1	2/3″ × 2½″ × ¼″ ⅃L (2/3½″ × 2½″ × 5⁄16″ ⅃L)	2/3½″ × 2½″ × 5⁄16″ ⅃L (2/4″ × 3″ × 5⁄16″ ⅃L)	2/3½″ × 2½″ × 5⁄16″ ⅃L (2/5″ × 3″ × 5⁄16″ ⅃L)	2/4″ × 3″ × 5⁄16″ ⅃L (2/5″ × 3″ × 5⁄16″ ⅃L)	12′-0″ Centres.
2	2/2½″ × 2″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	2/3″ × 2″ × ¼″ ⅃L	2/3″ × 2″ × ¼″ ⅃L	
3	2/2½″ × 2″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	
4	2/3″ × 2½″ × ¼″ ⅃L	2/3½″ × 2½″ × ¼″ ⅃L	2/3½″ × 2½″ × 5⁄16″ ⅃L	2/4″ × 3″ × 5⁄16″ ⅃L	
5	1/2½″ × 2″ × ¼″ L	1/2½″ × 2″ × ¼″ L	1/2½″ × 2″ × ¼″ L	1/2½″ × 2″ × ¼″ L	
6	1/3″ × 3″ × ¼″ L	1/3″ × 3″ × 5⁄16″ L	2/2½″ × 2″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	
7	1/2½″ × 2″ × ¼″ L	1/2½″ × 2″ × ¼″ L	1/2½″ × 2″ × ¼″ L	1/2½″ × 2″ × ¼″ L	
8	1/3″ × 3″ × ¼″ L	1/3½″ × 3½″ × 5⁄16″ L	2/3″ × 2″ × ¼″ ⅃L	2/3″ × 2″ × ¼″ ⅃L	
9	2/3″ × 3″ × ¼″ ┐└	2/3½″ × 3½″ × 5⁄16″ ┐└	2/3½″ × 3½″ × 5⁄16″ ┐└	2/4″ × 4″ × 5⁄16″ ┐└	
1	2/3″ × 2½″ × 5⁄16″ ⅃L (2/3½″ × 2½″ × ⅜″ ⅃L)	2/3½″ × 2½″ × 5⁄16″ ⅃L (2/5″ × 3″ × 5⁄16″ ⅃L)	2/3½″ × 2½″ × ⅜″ ⅃L (2/5″ × 3″ × 5⁄16″ ⅃L)	2/4″ × 3″ × ⅜″ ⅃L (2/5″ × 3″ × ⅜″ ⅃L)	15′-0″ Centres.
2	2/3″ × 2″ × ¼″ ⅃L	2/3″ × 2″ × ¼″ ⅃L	2/3″ × 2″ × 5⁄16″ ⅃L	2/3″ × 2″ × 5⁄16″ ⅃L	
3	2/2½″ × 2″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	
4	2/3″ × 2½″ × ¼″ ⅃L	2/3½″ × 2½″ × ¼″ ⅃L	2/3½″ × 2½″ × 5⁄16″ ⅃L	2/4″ × 3″ × 5⁄16″ ⅃L	
5	1/2½″ × 2″ × ¼″ L	1/2½″ × 2″ × ¼″ L	1/2½″ × 2″ × ¼″ L	1/2½″ × 2″ × ¼″ L	
6	1/3″ × 3″ × ¼″ L	1/3″ × 3″ × 5⁄16″ L	2/2½″ × 2″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	
7	1/2½″ × 2″ × ¼″ L	1/2½″ × 2″ × ¼″ L	1/2½″ × 2″ × ¼″ L	1/2½″ × 2″ × ¼″ L	
8	1/3″ × 3″ × 5⁄16″ L	1/3½″ × 3½″ × 5⁄16″ L	2/3″ × 2″ × ¼″ ⅃L	2/3″ × 2½″ × ¼″ ⅃L	
9	2/3″ × 3″ × 5⁄16″ ┐└	2/3½″ × 3½″ × 5⁄16″ ┐└	2/3½″ × 3½″ × ⅜″ ┐└	2/4″ × 4″ × ⅜″ ┐└	

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Sections in brackets for Member " 1 " allow for flexural stress due to purlin in addition to " principal " stress.

STANDARD SECTIONS FOR SIDE STANCHEONS.

Type 2.—The maximum bending moment occurs at the ground-level, the unit amount—ignoring effect of vertical load—being found as follows :—

$$\left.\begin{array}{ll}\text{Vertical load } W^2 & =128{\cdot}475 \\ \text{,, \quad ,, } W^1 & = 20\end{array}\right\} 148{\cdot}475 \text{ lbs. (total).}$$

HN = 24·627 lbs.

Leverage of HN = 4′-0″.

Proportion of HP^S = 20 lbs.

Leverage of HP^S = 4′-0″.

$BM=(HN+HP^S)\times 4'\text{-}0''=(24{\cdot}627+20)\times 48''=2142{\cdot}1$ inch-lbs.

Type 3.—The maximum bending moment occurs at the connection of the stay to the stancheon, the unit amount—ignoring effect of vertical load—being found as follows :—

Vertical load = 134·93 lbs.

Horizontal reaction at foot = 49·627 lbs.

Leverage of do. = 3′-0″.

$BM=49{\cdot}627\times 36''=1786{\cdot}57$ inch-lbs.

Type 4.—The maximum bending moment will be assumed as situated at the ground-level, and the unit amount—ignoring effect of vertical load—may be found as follows :—

Vertical load = 132·94 lbs.

Horizontal reaction at point of contraflexure = 42·127.

Leverage of do. = 1½ ft.

Side wind on portion below point of contraflexure = 1½ × 20 = 30 lbs.

Leverage of do. = ·75 feet.

$BM=(42{\cdot}127\times 18'')+(30\times 9'')=758{\cdot}286+270=1028{\cdot}286$ inch-lbs.

The actual vertical load for the stancheons may be obtained by the formula "$US\times\frac{C}{12}\times D$"; therefore the multipliers will be similar to those used for the principals. With regard to the multipliers for the unit bending moment, owing to the ratio $\frac{C}{12}$ in the above formula applying to the leverage as well as to the amount of the horizontal force which sets up bending action, this formula will be :—

$S=US\times\left(\frac{C}{12}\right)^2\times D$, and the various values are tabulated hereafter.

Span	Height	$\left(\frac{C}{12}\right)^2 \times D$ equals for	
		12'-0" centres	15'-0" centres
40'-0"	13'-4"	133·33	166·66
50'-0"	16'-8"	208·33	260·4
60'-0"	20'-0"	300	375
70'-0"	23'-4"	408·33	510·4
80'-0"	26'-8"	533·33	666·66
90'-0"	30'-0"	675	843·75
100'-0"	33'-4"	833·33	1041·66

The total stresses and the sections recommended for use are tabulated hereafter, and the calculations have been made in accordance with the assumption that the safe crippling stress may be taken as equal to that for a strut fixed at each end, and the radius of gyration taken about an axis in a direction parallel to the flanges, the safe crippling stress per square inch—6½ tons—being reduced by the "Claxton Fidler" formula for struts to the allowable value for the ratio $\frac{L}{R}$. This assumption necessitates the location of the side rails relative to the stancheons in such a manner as to stiffen the stancheon in the direction of the least radius of gyration.

To increase the utility of the standard stresses and sections, they should be adaptable for sheds of heights other than those fixed. This elasticity may be obtained somewhat as follows :—

Type 2.—The vertical load being of minor importance, the variable effect may be ignored and the section for any height may be approximated as follows :—

By reference to foregoing data :—

$$\left.\begin{array}{l}\text{Effect of span}=55\%\ \text{BM}\\ \text{,,}\quad\text{,, height}=45\%\ \text{BM}\end{array}\right\}\ \text{for standard height.}$$

If S =span of shed in question,

S^1 =span of shed used as basis.

BM=bending moment for stancheon (of similar height) used as basis.

M=required BM for stancheon in question.

$$\text{Then } M=\left[45\%+\left(55\%\times\frac{S}{S^1}\right)\right]\ \text{BM}.$$

Example.—Determine section of stancheon for 60′-0″ span shed, Type 2, 30′-0″ high, 12′-0″ centres.

$$M=\left[45\%+\left(55\%\times\frac{S}{S^1}\right)\right] BM=81{\cdot}66 \text{ BM}$$

$$=81{\cdot}66\times1{,}445{,}917=1{,}184{,}832 \text{ inch-lbs.}$$

∴ use 16″ × 6″ × 62 lbs. RSJ as per tabulated list of stancheons.

Type 3.—If the height (H^1) to span is 4 : 12 (standard ratio for Type 3), use tabulated section for Type 3.

If the height (H^1) to span is 2·5 : 12 (standard ratio for Type 4), use tabulated section for Type 4.

Stancheons for H^1 between 2·5 and 4 may be determined by comparison of the sections for these ratios.

Type 4.—If the height (H^1) to span is 2·5 : 12 (standard ratio for Type 4), use tabulated section for Type 4.

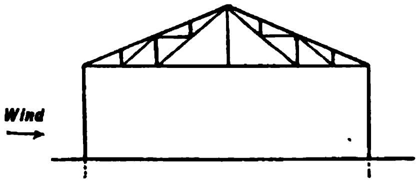
If the height (H^1) to span is 4 : 12 (standard ratio for Type 3), use tabulated section for Type 3.

Other ratios between 2·5 and 4 may be dealt with by comparison, as before described.

STANDARD SECTIONS FOR SIDE STANCHEONS.

TYPE 2.

SHEETED ROOFS.



Type 2				**Stancheons 12′-0″ Centres**				
Span of Shed	Height of Shed	Unit vert. load	Unit B.M.	Multipliers		Total V.L.	Total B.M.	I Section Required
				V.L.	B.M.			
		lbs.	inch-lbs.			lbs.	inch-lbs.	lbs.
40′-0″	13′-4″	148·475	2142·1	40	133	5939	284899	8″ × 5″ × 28
50′-0″	16′-8″	„	„	50	208	7424	445557	12″ × 5″ × 32
60′-0″	20′-0″	„	„	60	300	8908	642630	12″ × 6″ × 44
70′-0″	23′-4″	„	„	70	408	10393	873977	12″ × 6″ × 54
80′-0″	26′-8″	„	„	80	533	11878	1141739	16″ × 6″ × 62
90′-0″	30′-0″	„	„	90	675	13363	1445917	18″ × 7″ × 75
100′-0″	33′-4″	„	„	100	833	14847	1784369	20″ × 7½″ × 89
Type 2				**Stancheons 15′-0″ Centres**				
40′-0″	13′-4″	„	„	50	167	7424	357731	10″ × 5″ × 30
50′-0″	16′-8″	„	„	62·5	260	9280	556946	12″ × 5″ × 39
60′-0″	20′-0″	„	„	75	375	11136	803288	14″ × 6″ × 46
70′-0″	23′-4″	„	„	87·5	510	12992	1092471	15″ × 6″ × 59
80′-0″	26′-8″	„	„	100	667	14848	1428781	18″ × 7″ × 75
90′-0″	30′-0″	„	„	112·5	844	16704	1807932	20″ × 7½″ × 89
100′-0″	33′-4″	„	„	125	1042	18560	2232068	20″ × 7½″ × 89

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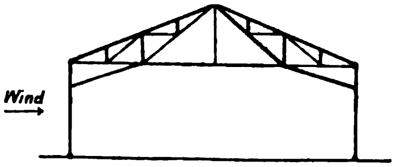
STANDARD SECTIONS FOR SIDE STANCHEONS.

TYPE 3.

SHEETED ROOFS.



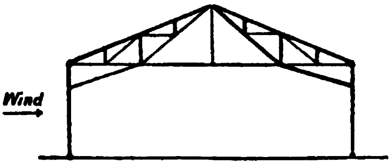
Span of Shed	Height of Shed	Unit vert. load	Unit B.M.	Multipliers		Total V.L.	Total B.M.	I Section Required
				V.L.	B.M.			
Type 3				**Stancheons 12′-0″ Centres**				
		lbs.	inch-lbs.			lbs.	inch-lbs.	lbs.
40′-0″	13′-4″	134·93	1786·57	40	133	5397	237614	9″ × 4″ × 21
50′-0″	16′-8″	”	”	50	208	6747	371606	10″ × 5″ × 30
60′-0″	20′-0″	”	”	60	300	8096	535971	12″ × 5″ × 39
70′-0″	23′-4″	”	”	70	408	9445	728920	14″ × 6″ × 46
80′-0″	26′-8″	”	”	80	533	10795	952242	14″ × 6″ × 57
90′-0″	30′-0″	”	”	90	675	12144	1205935	16″ × 6″ × 62
100′-0″	33′-4″	”	”	100	833	13493	1488213	18″ × 7″ × 75
Type 3				**Stancheons 15′-0″ Centres**				
40′-0″	13′-4″	”	”	50	167	6747	298357	8″ × 5″ × 28
50′-0″	16′-8″	”	”	62·5	260	8433	464508	12″ × 5″ × 32
60′-0″	20′-0″	”	”	75	375	10120	669964	12″ × 6″ × 44
70′-0″	23′-4″	”	”	87·5	510	11806	911150	14″ × 6″ × 57
80′-0″	26′-8″	”	”	100	667	13493	1191642	16″ × 6″ × 62
90′-0″	30′-0″	”	”	112·5	844	15180	1507865	18″ × 7″ × 75
100′-0″	33′-4″	”	”	125	1042	16866	1861605	20″ × 7½″ × 89

Sheet 2/20

STANDARD SECTIONS FOR SIDE STANCHEONS.

TYPE 4.

SHEETED ROOFS.



Span of Shed	Height of shed	Unit vert. load	Unit B.M.	Multipliers		Total V.L.	Total B.M.	I Section Required
				V.L.	B.M.			
Type 4				**Stancheons 12′-0″ Centres**				
		lbs.	inch-lbs.			lbs.	inch-lbs.	lbs.
40′-0″	13′-4″	132·94	1028·28	40	133	5318	136761	8″×4″×18
50′-0″	16′-8″	”	”	50	208	6647	213882	9″×4″×21
60′-0″	20′-0″	”	”	60	300	7976	308484	8″×5″×28
70′-0″	23′-4″	”	”	70	408	9306	419538	12″×5″×32
80′-0″	26′-8″	”	”	80	533	10635	548073	12″×5″×39
90′-0″	30′-0″	”	”	90	675	11965	694089	12″×6″×44
100′-0″	33′-4″	”	”	100	833	13294	856557	12″×6″×54
Type 4				**Stancheons 15′-0″ Centres**				
40′-0″	13′-4″	”	”	50	167	6647	171723	9″×4″×21
50′-0″	16′-8″	”	”	62·5	260	8308	267353	8″×5″×28
60′-0″	20′-0″	”	”	75	375	9971	385605	10″×5″×30
70′-0″	23′-4″	”	”	87·5	510	11633	524423	12″×5″×39
80′-0″	26′-8″	”	”	100	667	13294	685863	12″×6″×44
90′-0″	30′-0″	”	”	112·5	844	14956	867868	12″×6″×54
100′-0″	33′-4″	”	”	125	1042	16618	1071468	15″×6″×59

Sheet 2/21

STANDARD SECTIONS FOR SIDE RAILS.

The forces acting on the side rails are those due to wind, to the weight of the side covering, and to the weight of the rail, the amounts being as follows :—

Wind : 20 lbs. per sup. ft.
Covering : 3 " "
Rail : 5 to 11 lbs. per lin. ft. according to the span.

Assuming a constant spacing of 6′-6″, then the above forces per foot run are :—

Wind : 6′-6″ × 20 lbs. = 130 lbs.
Covering : 6′-6″ × 3 " = 19·5 "
Rail : 5 to 11 lbs.

Calculations for rail 12′-0″ span.

$$\text{BM due to wind}=\frac{WL}{8}=\frac{130\times12\times144}{8}=28{,}080 \text{ inch-lbs.}$$

MR = BM = 28,080 inch-lbs.
MS = MR ÷ F, and as F = 22,400 lbs., then

$$MS=\frac{28{,}080}{22{,}400}=1{\cdot}25.$$

Nearest British standard joist section is 4″ × 1¾″ × 5 lbs. with a maximum MS = 1·83.

$$\therefore \text{ actual stress}=\frac{28{,}080}{1{\cdot}83}=15{,}344 \text{ lbs. per sq. inch.}$$

Assuming the 4″ × 1¾″ I rail is supported at its centre by means of a sag rod, then the stress due to the vertical load may be found as follows :—

Weight of side sheeting = 19·5 × 6′-0″ = 117 lbs.
" " rail = 5 × 6′-0″ = 30 "
147 "

$$BM=\frac{WL}{8}=\frac{147\times72}{8}=1323 \text{ inch-lbs.}$$

Minimum MS = ·22.

$$\text{Stress per sq. inch}=\frac{1323}{{\cdot}22}=6013 \text{ lbs.}$$

Total stress per sq. inch = 15,344 + 6013 = 21,357 lbs., which is within the safe limits of stress.

Calculations for rail 15′-0″ span :—

$$\text{BM due to wind}=\frac{WL}{8}=\frac{130\times15\times180}{8}=43{,}875 \text{ inch-lbs.}$$

$$MS=\frac{43{,}875}{22{,}400}=1{\cdot}96.$$

Assuming a 4¾″×1¾″×6½ lbs. RSJ as suitable, then maximum MS=2·85 and

$$\text{actual stress}=\frac{43{,}875}{2{\cdot}85}=15{,}394 \text{ lbs. per sq. inch.}$$

Weight due to vertical load equals :—

Weight of side sheeting $=19{\cdot}5\times7{\cdot}5=146{\cdot}25$ lbs.
„ „ rail $=\ 6{\cdot}5\times7{\cdot}5=\ \underline{48{\cdot}75}$ „
$\underline{195}$ „

$$\text{BM}=\frac{\text{WL}}{8}=\frac{195\times90}{8}=2194 \text{ inch-lbs.}$$

Minimum MS=·3.

$$\text{Stress per sq. inch}=\frac{2194}{{\cdot}3}=7313 \text{ lbs.}$$

Total stress per sq. inch=15,394+7313=22,707 lbs., therefore the 4¾″×1¾″ RSJ is suitable.

CALCULATIONS FOR RAIL 16′-8″ SPAN.

$$\text{BM due to wind}=\frac{\text{WL}}{8}=\frac{130\times16{\cdot}66\times200}{8}=54{,}166 \text{ inch-lbs.}$$

$$\text{MS}=\frac{54{,}166}{22{,}400}=2{\cdot}42.$$

Assuming a 5″×3″×11 lbs. RSJ as suitable, then Maximum MS=5·4 and

$$\text{Actual stress}=\frac{54{,}166}{5{\cdot}4}=10{,}030 \text{ lbs. per sq. inch.}$$

Weight due to vertical load equals :—

Weight of side sheeting $=19{\cdot}5\times8{\cdot}3\dot{3}=162{\cdot}5$ lbs.
„ „ rail $=11\ \times8{\cdot}3\dot{3}=\ \underline{91{\cdot}6}$ „
$\underline{254{\cdot}1}$ „

$$\text{BM}=\frac{\text{WL}}{8}=\frac{254\times100}{8}=3175 \text{ inch-lbs.}$$

Minimum MS=·975.

$$\text{Stress per sq. inch}=\frac{3175}{{\cdot}975}=3256 \text{ lbs.}$$

Total stress per sq. inch=10,030+3256=13,286 lbs.

Although the actual stress is much less than the allowable stress, the section is the nearest suitable standard section, and will therefore be used.

STANDARD SECTIONS FOR ROOF PURLINS.

The forces acting on the purlins are those due to wind, to the weight of the purlins, and to the weight of the covering, the amounts being as follows :—

Wind : Normal pressure = 18 lbs. per sup. ft.
Covering : 3 lbs. per sup. ft.
Purlin : 5 to 6½ lbs. per ft. run.

These forces and also the effect being slightly less than those for the rails, the sections adopted for the latter may be used here, i.e. :—

4″ × 1¾″ × 5 lbs. RSJ purlins for spans up to 12′-0″.
4¾″ × 1¾″ × 6½ „ „ „ „ 12′-0″ to 15′-0″.

End Framing.—The determination of the stresses in the various members is somewhat complex, consequently it is not practicable, nor is it necessary, to standardize the stresses as for the other portions of the shed framework. By making certain approximations, however, as described in detail previously at page 14 *et seq.*, it is possible to arrive at suitable sections which are tabulated on sheets 2/22 and 2/23 for 40 to 60 ft. spans, and on sheets 2/24 and 2/25 for 70 to 100 ft. spans.

For purposes of economy, etc., it is advisable—for 70 to 100 ft. spans—to provide a wind girder between the gable and the first principal or transverse frame, in the plane of the bottom chord. The tensile stress in that flange of the wind girder coincident with the bottom chord of the principal, supplements that in the latter member, and, as the two stresses have been allowed for in calculating the section of the girder flange, the latter will act dually as the girder flange and as the bottom chord of the principal.

By utilizing the purlins and rails to resist—in addition to the bending stresses—the compressive stresses in the struts, the latter members may, of course, be dispensed with, if desired.

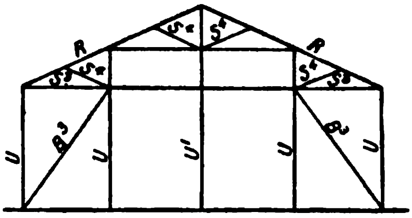
Owing to the great length of the braces, especially for the longer spans, they should be suitably connected to the adjacent purlins and rails.

Foundations.—The sizes of side and gable concrete foundations for the various types and spans are given on sheet 2/27, page 61, and have been calculated in the case of Types 2 and 4 for a safe earth pressure of 3 tons per sq. ft., and for Type 3 of 1 ton per sq. ft.

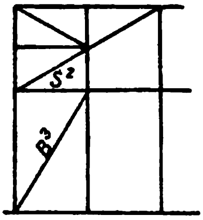
The safe pressure adopted for Types 2 and 4, i.e. 3 tons per sq. ft., is a fairly high value, especially as on the leeward side the maximum pressure occurs at the surface of the ground, and unless the earth is very compact and solid, or is aided by a floor of concrete or other rigid substance, the resistance offered may be insufficient to properly fix the stancheon, and these types should then for economy and efficiency be superseded by Type 3, or in the case of Type 4 only the base may be considered as partially fixed, and the stancheon and foundation designed accordingly.

GABLE FRAMES AND BRACINGS
TO SUIT PRINCIPALS 12 FT. CENTRES.
SECTIONS FOR 40 FT. TO 60 FT. SPAN.

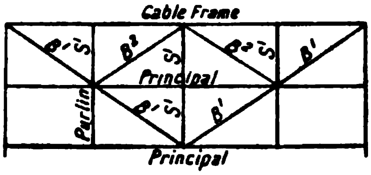
SHEETED ROOFS.



Front Elevation.



Side Elevation.



Plan.

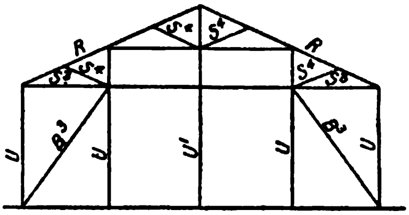
NOTE.—Rails and purlins to be spaced not greater than 6′-6″ apart. Sections as follows:—Up to 12′-0″ span, 4″ × 1¾″ × 5 lbs. I; 12′-6″ to 15′-0″ spans, 4¾″ × 1¾″ × 6½ lbs. I.

MEMBER		40′-0″ SPAN	50′-0″ SPAN	60′-0″ SPAN
Upright	U	9″ × 4″ × 21 lbs. I	10″ × 5″ × 30 lbs. I	12″ × 5″ × 32 lbs. I
„	U¹	10″ × 5″ × 30 lbs. I	12″ × 5″ × 32 lbs. I	12″ × 6″ × 44 lbs. I
Rafter	R	1/4″ × 2″ ⊓	1/5½″ × 2″ ⊓	1/6″ × 2½″ ⊓
Strut	S¹	2/3″ × 2″ × ¼″ ⅃L	2/3″ × 2″ × ¼″ ⅃L	2/3″ × 2½″ × ¼″ ⅃L
„	S²	2/3″ × 2″ × ¼″ ⅃L	2/3″ × 2″ × ¼″ ⅃L	2/3″ × 2″ × ¼″ ⅃L
„	S³	2/3″ × 2″ × ¼″ ⅃L	2/3″ × 2″ × ¼″ ⅃L	2/3″ × 2″ × ¼″ ⅃L
„	S⁴	1/2½″ × 2½″ × ¼″ L	1/2½″ × 2½″ × ¼″ L	1/3″ × 3″ × ¼″ L
Brace	B¹	1/2½″ × 2″ × ¼″ L	1/3″ × 2″ × ¼″ L	1/3″ × 3″ × ¼″ L
„	B²	1/2½″ × 2″ × ¼″ L	1/3″ × 2″ × ¼″ L	1/3″ × 2″ × ¼″ L
„	B³	1/3½″ × ⅜″ Flat	1/3½″ × 7/16″ Flat	1/3½″ × ½″ Flat

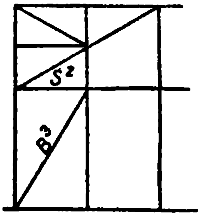
SHEET 2/22

SECTIONS FOR 40 FT. TO 60 FT. SPAN GABLE FRAMES AND BRACINGS TO SUIT PRINCIPALS 15 FT. CENTRES.

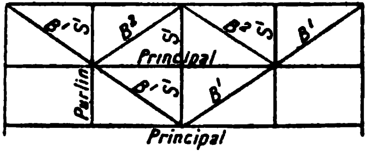
SHEETED ROOFS.



Front Elevation.



Side Elevation.



Plan.

Note.—Rails and purlins to be spaced not greater than 6′-6″ apart. Sections as follows:—Up to 12′-0″ span, 4″ × 1¾″ × 5 lbs. I; 12′-6″ to 15′-0″ spans, 4¾″ × 1¾″ × 6½ lbs. I.

Member	40′-0″ Span	50′-0″ Span	60′-0″ Span
Upright U	9″ × 4″ × 21 lbs. I	10″ × 5″ × 30 lbs. I	12″ × 5″ × 32 lbs. I
„ U^1	10″ × 5″ × 30 lbs. I	12″ × 5″ × 32 lbs. I	12″ × 6″ × 44 lbs. I
Rafter R	1/4″ × 2″ ⊓	1/5″ × 2½″ ⊓	1/6″ × 2½″ ⊓
Strut S^1	2/3½″ × 2½″ × ¼″ ⅃L	2/3½″ × 2½″ × ¼″ ⅃L	2/3½″ × 2½″ × ¼″ ⅃L
„ S^2	2/3½″ × 2½″ × ¼″ ⅃L	2/3½″ × 2½″ × ¼″ ⅃L	2/3½″ × 2½″ × ¼″ ⅃L
„ S^3	2/3½″ × 2½″ × ¼″ ⅃L	2/3½″ × 2½″ × ¼″ ⅃L	2/3½″ × 2½″ × ¼″ ⅃L
„ S^4	1/2½″ × 2½″ × ¼″ L	1/2½″ × 2½″ × ¼″ L	1/3″ × 3″ × ¼″ L
Brace B^1	1/2½″ × 2″ × ¼″ L	1/3″ × 2″ × ¼″ L	1/3″ × 3″ × ¼″ L
„ B^2	1/2½″ × 2″ × ¼″ L	1/3″ × 2″ × ¼″ L	1/3″ × 2″ × ¼″ L
„ B^3	1/3½″ × ⅜″ Flat	1/3½″ × 7/16″ Flat	1/3½″ × ½″ Flat

Sheet 2/23

SECTIONS FOR 70′-0″ TO 100′-0″ SPAN GABLE FRAMES AND BRACINGS TO SUIT PRINCIPALS 12′-0″ CENTRES.

For outline diagram see Sheet 2/26, page 60.

SHEETED ROOFS.

MEMBER	70′-0″ SPAN	80′-0″ SPAN	90′-0″ SPAN	100′-0″ SPAN
Upright U	10″ × 5″ × 30 lbs. I	12″ × 5″ × 32 lbs. I	14″ × 6″ × 46 lbs. I	14″ × 6″ × 57 lbs. I
„ U^1	7″ × 4″ × 16 lbs. I	7″ × 4″ × 16 lbs. I	8″ × 4″ × 18 lbs. I	9″ × 4″ × 21 lbs. I
„ U^2	9″ × 4″ × 21 lbs. I	9″ × 4″ × 21 lbs. I	10″ × 5″ × 30 lbs. I	10″ × 5″ × 30 lbs. I
Brace B^3	1/4″ × 4″ × ⅜″ └	1/4″ × 4″ × $\tfrac{7}{16}$″ └	1/4″ × 4″ × ½″ └	1/5″ × 4″ × ½″ └
Strut S^2	2/3½″ × 2½″ × ¼″ ┘└ S	2/3½″ × 3½″ × $\tfrac{5}{16}$″ ┘└ S	2/3½″ × 3″ × $\tfrac{5}{16}$″ ┘└ S	2/4″ × 3″ × $\tfrac{5}{16}$″ ┘└ S
„ S^3	2/4″ × 3″ × ⅜″ ┘└ S	2/5″ × 3″ × ⅜″ ┘└ S	2/5″ × 4″ × ⅜″ ┘└ S	2/6″ × 4″ × ½″ ┘└ S
„ S^4	1/3″ × 3″ × ¼″ └	1/3″ × 3″ × $\tfrac{5}{16}$″ └	1/3½″ × 3½″ × $\tfrac{5}{16}$″ └	1/4″ × 4″ × ⅜″ └
Rafter R	7″ × 3″ × 17½ lbs. ⊓	7″ × 3″ × 17½ lbs. ⊓	9″ × 3″ × 19½ lbs. ⊓	9″ × 3″ × 19½ lbs. ⊓
Strut S	2/3″ × 2″ × ¼″ ┘└ S	2/3″ × 2″ × ¼″ ┘└ S	2/4″ × 2″ × ¼″ ┘└ S	2/4″ × 2″ × ¼″ ┘└ S
„ S^1	2/3″ × 2″ × ¼″ ┘└ S	2/3″ × 2″ × ¼″ ┘└ S	2/4″ × 2″ × ¼″ ┘└ S	2/4″ × 2″ × ¼″ ┘└ S
Brace B^1	1/3″ × 3″ × ¼″ └	1/3″ × 3″ × $\tfrac{5}{16}$″ └	1/4″ × 3″ × $\tfrac{5}{16}$″ └	1/5″ × 3″ × $\tfrac{5}{16}$″ └
„ B^2	1/3″ × 3″ × ¼″ └	1/3″ × 3″ × $\tfrac{5}{16}$″ └	1/4″ × 3″ × $\tfrac{5}{16}$″ └	1/5″ × 3″ × $\tfrac{5}{16}$″ └
„ T^1	1/4″ × 3″ × $\tfrac{5}{16}$″ └	1/4″ × 3″ × $\tfrac{5}{16}$″ └	1/4½″ × 3″ × $\tfrac{5}{16}$″ └	1/5″ × 3″ × $\tfrac{5}{16}$″ └
Strut S^5	2/5″ × 3″ × $\tfrac{5}{16}$″ ┘└ S	2/5″ × 3″ × ⅜″ ┘└ S	2/5″ × 3½″ × ⅜″ ┘└ S	2/6″ × 4″ × ⅜″ ┘└ S
„ S^6	2/3″ × 2″ × ¼″ ┘└ S	2/3″ × 2″ × ¼″ ┘└ S	2/3″ × 2″ × ¼″ ┘└ S	2/3½″ × 2″ × ¼″ ┘└ S
„ S^7	2/3″ × 2½″ × $\tfrac{5}{16}$″ ┘└ S	2/3½″ × 3″ × $\tfrac{5}{16}$″ ┘└ S	2/4″ × 3″ × $\tfrac{5}{16}$″ ┘└ S	2/5″ × 3″ × $\tfrac{5}{16}$″ ┘└ S
Flange F	2/4″ × 3″ × ⅜″ ┘└ S	2/5″ × 3″ × ⅜″ ┘└ S	2/5″ × 4″ × ⅜″ ┘└ S	2/6″ × 4″ × ½″ ┘└ S
„ F^1	2/3″ × 2½″ × $\tfrac{5}{16}$″ ┘└ S	2/3½″ × 2½″ × $\tfrac{5}{16}$″ ┘└ S	2/4″ × 3″ × $\tfrac{5}{16}$″ ┘└ S	2/5″ × 3″ × ⅜″ ┘└ S

SHEET 2/24

SECTIONS FOR 70′-0″ TO 100′-0″ SPAN GABLE FRAMES AND BRACINGS TO SUIT PRINCIPALS 15′-0″ CENTRES.

For outline diagram see Sheet 2/26, page 60.

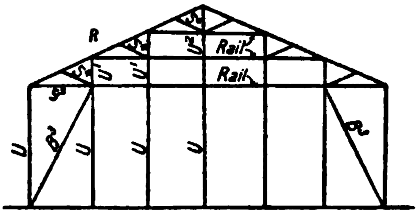
SHEETED ROOFS.

Member		70′-0″ Span	80′-0″ Span	90′-0″ Span	100′-0″ Span
Upright	U	10″×5″×30 lbs. I	12″×5″×32 lbs. I	14″×6″×46 lbs. I	14″×6″×57 lbs. I
,,	U^1	7″×4″×16 lbs. I	7″×4″×16 lbs. I	8″×4″×18 lbs. I	9″×4″×21 lbs. I
,,	U^2	9″×4″×21 lbs. I	9″×4″×21 lbs. I	10″×5″×30 lbs. I	10″×5″×30 lbs. I
Brace	B^3	1/4″×4″×⅜″ ⌊	1/4″×4″×⁷⁄₁₆″ ⌊	1/4″×4″×½″ ⌊	1/5″×4″×½″ ⌊
Strut	S^2	2/3½×2½″×⁵⁄₁₆″ ⌋⌊S	2/3½″×3″×⁵⁄₁₆″ ⌋⌊S	2/4″×3″×⁵⁄₁₆″ ⌋⌊S	2/5″×3″×⁵⁄₁₆″ ⌋⌊S
,,	S^3	2/4″×3″×⅜″ ⌋⌊S	2/5″×3″×⅜″ ⌋⌊S	2/5″×4″×⅜″ ⌋⌊S	2/6″×4″×½″ ⌋⌊S
,,	S^4	1/3″×3″×¼″ ⌊	1/3″×3″×⁵⁄₁₆″ ⌊	1/3½″×3½″×⁵⁄₁₆″ ⌊	1/4″×4″×⅜″ ⌊
Rafter	R	7″×3″×17½ lbs. ⊓	7″×3″×17½ lbs. ⊓	9″×3″×19½ lbs. ⊓	9″×3″×19½ lbs. ⊓
Strut	S	2/3″×2½″×¼″ ⌋⌊S	2/3½″×2½″×¼″ ⌋⌊S	2/4″×3″×⁵⁄₁₆″ ⌋⌊S	2/4″×3″×⁵⁄₁₆″ ⌋⌊S
,,	S^1	2/3″×2½″×¼″ ⌋⌊S	2/3½″×2½″×¼″ ⌋⌊S	2/4″×3″×⁵⁄₁₆″ ⌋⌊S	2/4″×3″×⁵⁄₁₆″ ⌋⌊S
Brace	B^1	1/3″×3″×¼″ ⌊	1/3″×3″×⁵⁄₁₆″ ⌊	1/4″×3″×⁵⁄₁₆″ ⌊	1/5″×3″×⁵⁄₁₆″ ⌊
,,	B^2	1/3″×3″×¼″ ⌊	1/3″×3″×⁵⁄₁₆″ ⌊	1/4″×3″×⁵⁄₁₆″ ⌊	1/5″×3″×⁵⁄₁₆″ ⌊
,,	T^1	1/4″×3″×⁵⁄₁₆″ ⌊S	1/4″×3″×⁵⁄₁₆″ ⌊	1/4½″×3″× ⁵⁄₁₆″ ⌊	1/5″×3″×⁵⁄₁₆″ ⌊
Strut	S^5	2/6″×3″×⅜″ ⌋⌊S	2/6″×3½″×⅜″ ⌋⌊S	2/6″×3½″×⁷⁄₁₆″ ⌋⌊S	2/6″×4″×⁷⁄₁₆″ ⌋⌊S
,,	S^6	2/3″×2½″×¼″ ⌋⌊S	2/3″×2½″×¼″ ⌋⌊S	2/3″×2½″×¼″ ⌋⌊S	2/3½″×2½″×¼″ ⌋⌊S
,,	S^7	2/4″×3″×⁵⁄₁₆″ ⌋⌊S	2/4″×3″×⁵⁄₁₆″ ⌋⌊S	2/5″×3″×⁵⁄₁₆″ ⌋⌊S	2/6″×3″×⁵⁄₁₆″ ⌋⌊S
Flange	F	2/4″×3″×⅜″ ⌋⌊S	2/5″×3″×⅜″ ⌋⌊S	2/5″×4″×⅜″ ⌋⌊S	2/6″×4″×½″ ⌋⌊S
,,	F^1	2/3″×2½″×⁵⁄₁₆″ ⌋⌊S	2/3½″×2½″×⁵⁄₁₆″ ⌋⌊S	2/4″×3″×⁵⁄₁₆″ ⌋⌊S	2/5″×3″×⅜″ ⌋⌊S

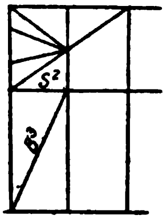
Sheet 2/25

OUTLINE DIAGRAM FOR GABLE FRAMES FOR SPANS 70'-0" TO 100'-0".

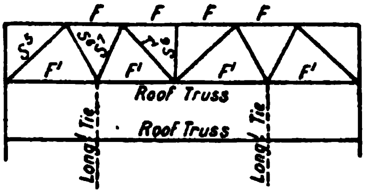
SHEETED ROOFS.



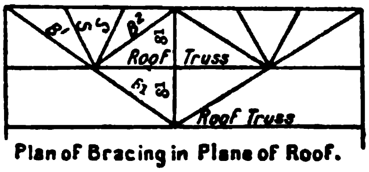
Front Elevation.



Side Elevation.



Plan of Bracing in Plane of Bottom Chord.



Plan of Bracing in Plane of Roof.

Note.—Rails and Purlins to be spaced not more than 6'-6" apart.

Sections as follows:—

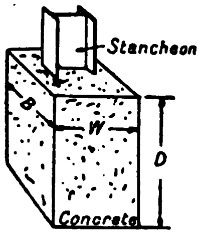
Up to 12'-0" span, 4"×1¾"×5 lbs. I.
12'-6" to 15'-0", 4¾"×1¾"×6½ „ I.
15'-0" to 16'-8", 5"×3"×11 „ I.

CONCRETE FOUNDATIONS FOR SHEETED SHEDS.

Basis : Earth pressure for Types 2 and 4 : 3 tons per sq. ft.
,, ,, ,, Type 3 : 1 ton ,, ,,

Span	Height	Type 2	Type 3	Type 4
For Side Stanchions 12′-0″ Centres				
		D W B	B W D	D W B
40′-0″	13′-4″	3′-6″ × 2′-3″ × 2′-3″	2′-0″ × 2′-0″ × 1′-3″	2′-9″ × 2′-0″ × 2′-0″
50′-0″	16′-8″	4′-0″ × 2′-6″ × 2′-6″	2′-2″ × 2′-2″ × 1′-4″	3′-6″ × 2′-3″ × 2′-3″
60′-0″	20′-0″	4′-6″ × 2′-9″ × 2′-9″	2′-4″ × 2′-4″ × 1′-6″	4′-0″ × 2′-6″ × 2′-6″
70′-0″	23′-4″	5′-0″ × 3′-0″ × 3′-0″	2′-6″ × 2′-6″ × 1′-6″	4′-3″ × 2′-8″ × 2′-8″
80′-0″	26′-8″	5′-6″ × 3′-3″ × 3′-3″	2′-8″ × 2′-8″ × 1′-8″	4′-6″ × 2′-10″ × 2′-10″
90′-0″	30′-0″	6′-0″ × 3′-8″ × 3′-8″	2′-10″ × 2′-10″ × 1′-10″	4′-9″ × 3′-2″ × 3′-2″
100′-0″	33′-4″	6′-6″ × 4′-0″ × 4′-0″	3′-0″ × 3′-0″ × 2′-0″	5′-0″ × 3′-6″ × 3′-6″
For Side Stanchions 15′-0″ Centres				
40′-0″	13′-4″	3′-9″ × 2′-6″ × 2′-6″	2′-6″ × 2′-6″ × 1′-3″	3′-0″ × 2′-0″ × 2′-0″
50′-0″	16′-8″	4′-0″ × 3′-0″ × 3′-0″	2′-9″ × 2′-9″ × 1′-4″	3′-9″ × 2′-4″ × 2′-4″
60′-0″	20′-0″	4′-6″ × 3′-6″ × 3′-6″	3′-0″ × 3′-0″ × 1′-6″	4′-0″ × 3′-0″ × 3′-0″
70′-0″	23′-4″	5′-0″ × 3′-9″ × 3′-9″	3′-2″ × 3′-2″ × 1′-6″	4′-4″ × 3′-4″ × 3′-4″
80′-0″	26′-8″	5′-6″ × 4′-0″ × 4′-0″	3′-4″ × 3′-4″ × 1′-8″	4′-8″ × 3′-9″ × 3′-9″
90′-0″	30′-0″	6′-0″ × 4′-6″ × 4′-6″	3′-6″ × 3′-6″ × 1′-10″	5′-0″ × 4′-0″ × 4′-0″
100′-0″	33′-4″	6′-6″ × 5′-0″ × 5′-0″	3′-9″ × 3′-9″ × 2′-0″	5′-4″ × 4′-3″ × 4′-3″

Gable Stanchions		
		W B D
40′-0″	13′-4″	1′-6″ × 1′-6″ × 1′-3″
50′-0″	16′-8″	1′-9″ × 1′-9″ × 1′-6″
60′-0″	20′-0″	2′-0″ × 2′-0″ × 1′-6″
70′-0″	23′-4″	2′-0″ × 2′-0″ × 1′-6″
80′-0″	26′-8″	2′-0″ × 2′-0″ × 1′-6″
90′-0″	30′-0″	2′-3″ × 2′-3″ × 1′-9″
100′-0″	33′-4″	2′-6″ × 2′-6″ × 2′-0″



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CHAPTER III.

STANDARD SECTIONS FOR SHEDS WITH OPEN SIDES AND GABLES. SHEETED ROOFS.

The preceding chapters deal with sheds having the sides and gables entirely closed in, and are not therefore wholly applicable to sheds with open sides and gables; therefore the sizes of the various members will need reconsidering on the lines described hereafter.

Adopting the same symbols and data the force constants for, and amounts of, reactions will be as follows :—

Type 2, Principals (for $24\frac{1}{2}°$ pitch).

$$\text{Total upward reaction at WRP} = \frac{W}{2} + \cdot 637\ NP$$
$$= 52\cdot 8 + 75\cdot 675 = 128\cdot 475 \text{ lbs.}$$

$$\text{Total upward reaction at LRP} = \frac{W}{2} + \cdot 273\ NP$$
$$= 52\cdot 8 + 32\cdot 432 = 85\cdot 232 \text{ lbs.}$$

Total horizontal reaction at WRP and at LRP

$= \cdot 2073\ NP = 24\cdot 627$ lbs.

Type 3, Transverse Frames.

$$\text{Total upward reaction at WRT} = \frac{W}{2} + \left(\frac{C - (\cdot 3C + \cdot 4557S^2)}{C} \times VN\right)$$
$$= 52\cdot 8 + 59\cdot 2 = 112 \text{ lbs.}$$

$$\text{Total upward reaction at LRT} = \frac{W}{2} + \left(\frac{\cdot 3C + \cdot 4557S^2}{C} \times VN\right)$$
$$= 52\cdot 8 + 48\cdot 8 = 101\cdot 6 \text{ lbs.}$$

Total horizontal reaction at WRT and at LRT

$= \cdot 2073\ NP = 24\cdot 627$ lbs.

Type 4, Transverse Frames.

$$\text{Total upward reaction at WRT} = \frac{W}{2} + \left(\frac{C-(\cdot 3C+\cdot 4557S^2)}{C} \times VN\right)$$

$$= 52\cdot 8 + 65\cdot 34 = 118\cdot 14 \text{ lbs.}$$

$$\text{Total upward reaction at LRT} = \frac{W}{2} + \left(\frac{\cdot 3C+\cdot 4557S^2}{C} \times VN\right)$$

$$= 52\cdot 8 + 42\cdot 65 = 95\cdot 45 \text{ lbs.}$$

Total horizontal reaction at WRT and at LRT
$= \cdot 2073$ NP $= 24\cdot 627$ lbs.

The only difference in the stresses in the principal members for the two classes of sheds, i.e. open and closed sheds, is that in the bottom chord, such difference being due to the absence (in the open-shed type) of that portion of the side-wind force—acting on the closed-shed type—transmitted from the windward to the leeward side. The effect of this force being that of counteracting and therefore reducing the tensile force in the bottom chord, it follows that the total stress in the latter member may be found by adding this force to the tabulated stresses given in the preceding chapter, and, as the tabulated sections have for practical reasons been made sufficient for this purpose, the sections given for the closed type may be adopted for use in the open type of shed.

Side Stancheons.—The horizontal forces, and therefore the bending moments, are, of course, much less for the open type than for the closed type sheds.

The unit stresses for the various types are given hereafter.

Type 2.—The maximum bending moment occurs at the ground-level, the unit amount being found as follows :—

Maximum vertical load occurs at WRP and equals 128·475 lbs.
Horizontal reaction at WRP $= 24\cdot 627$ lbs.
Leverage of do. $= 4$ ft.
BM $= 24\cdot 627 \times 48'' = 1182\cdot 096$ inch-lbs.

Type 3.—The maximum bending moment occurs at the connection of the stay to the stancheon, the unit amount being found as follows :—

Maximum vertical load occurs at WRT and equals 112 lbs.
Horizontal reaction at WRT $= 24\cdot 627$ lbs.
Leverage of do. $= 3$ ft.
BM $= 24\cdot 627 \times 36'' = 886\cdot 572$ inch-lbs.

Type 4.—The maximum bending moment occurs at the connection of the stay to the stancheon, the unit amount being found as follows :—

Maximum vertical load occurs at WRT and equals 118·14 lbs.
Horizontal reaction at WRT = 24·627 lbs.
Leverage of do. = 1½ ft.
BM = 24·627 × 18″ = 443·286 inch-lbs.

The total stresses may be found by multiplying the unit stresses by the coefficients given previously. The total stresses, together with the sections recommended for use, are tabulated on Sheets 3/1, 3/2, and 3/3, page 66 *et seq.*, for Types 2, 3, and 4 respectively.

Roof Bracing.—The functions of the roof bracing are, the prevention of side flexure in the principal rafters and the stiffening of the framework during construction. For these purposes it is only necessary to use braces B^1 and B^2, and the section of these members may be similar to those adopted for the closed type of shed.

Side Braces.—The function of these members is to eliminate distortion in the transverse frames, due to the action of the horizontal forces acting at the reaction points WRT and LRT.

If S^1 = distance from WRP to connection of stay to the stancheon,
S^2 = distance of WRP from WRT.
RHP = horizontal force acting at WRT.
A = angle of stay to horizontal = 18½°.

Then the stress in the stay will be tensile at the windward side and compressive at the leeward side, and the amount will be :—

$$\frac{\text{RHP} \times S^2}{S^1} \times \text{Sec. A.}$$

The unit stress in the stay for Types 3 and 4 is as follows :—

Type 3.—

$$\frac{\text{RHP} \times S^2}{S^1} \times \text{Sec. A} = \frac{24{\cdot}627 \times 4}{1} \times 1{\cdot}054 = 103{\cdot}8 \text{ lbs.}$$

Type 4.—

$$\frac{\text{RHP} \times S^2}{S^1} \times \text{Sec. A} = \frac{24{\cdot}627 \times 2\tfrac{1}{2}}{1} \times 1{\cdot}054 = 64{\cdot}9 \text{ lbs.}$$

The total stress for the various spans may be obtained by multiplying the above unit stresses by the multipliers given in the preceding chapter for the roof members.

Although the unit stresses given above are much less than those for the closed type of shed, the sections given for the latter will apply here, as these sections were determined by the ratio of $\frac{L}{R}$, which, of course, remains the same.

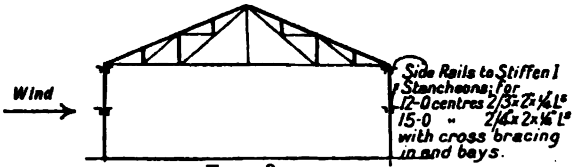
Roof Purlins.—The sections of the purlins will be exactly like those for the closed type of shed.

Side Foundations.—The sizes of the foundations for the various types and spans are given on Sheet 3/4, page 69, and have been calculated for the same earth pressure previously adopted.

STANDARD SECTIONS FOR SIDE STANCHEONS.

TYPE 2.

OPEN SHEDS—SHEETED ROOFS.



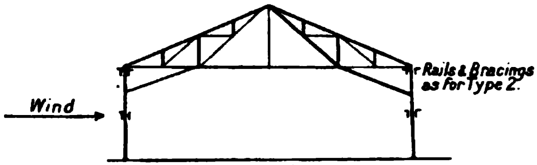
Type 2.

Span of Shed	Height of shed	Unit vert. load	Unit B.M.	Multipliers		Total V.L.	Total B.M.	I Section Required
Type 2				**Stancheons 12′-0″ Centres**				
				V.L.	B.M.			
		lbs.	inch-lbs.			lbs.	inch-lbs.	lbs.
40′-0″	13′-4″	129	1182	40	133	5160	157206	9″×4″×21
50′-0″	16′-8″	,,	,,	50	208	6450	245856	8″×5″×28
60′-0″	20′-0″	,,	,,	60	300	7740	354600	10″×5″×30
70′-0″	23′-4″	,,	,,	70	408	9030	482256	12″×5″×32
80′-0″	26′-8″	,,	,,	80	533	10320	630006	12″×6″×44
90′-0″	30′-0″	,,	,,	90	675	11610	797850	14″×6″×46
100′-0″	33′-4″	,,	,,	100	833	12900	984606	14″×6″×57
Type 2				**Stancheons 15′-0″ Centres**				
40′-0″	13′-4″	,,	,,	50	167	6450	197394	9″×4″×21
50′-0″	16′-8″	,,	,,	62·5	260	8062	307320	12″×5″×32
60′-0″	20′-0″	,,	,,	75	375	9675	443250	12″×5″×32
70′-0″	23′-4″	,,	,,	87·5	510	11287	602820	12″×6″×44
80′-0″	26′-8″	,,	,,	100	667	12900	788394	14″×6″×46
90′-0″	30′-0″	,,	,,	112·5	844	14512	997608	14″×6″×57
100′-0″	33′-4″	,,	,,	125	1042	16125	1231644	16″×6″×62

STANDARD SECTIONS FOR SIDE STANCHEONS.

TYPE 3.

OPEN SHEDS—SHEETED ROOFS.



Type 3.

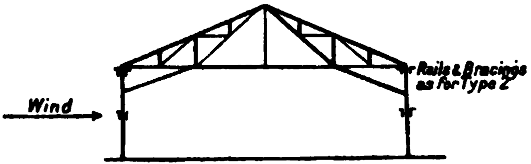
Type 3 **Stancheons 12′-0″ Centres**

Span of Shed	Height of Shed	Unit vert. load	Unit B.M.	Multipliers		Total V.L.	Total B.M.	I Section Required
				V.L.	B.M.			
		lbs.	inch-lbs.			lbs.	inch-lbs.	lbs.
40′-0″	13′-4″	112	887	40	133	4480	117971	7″×4″×16
50′-0″	16′-8″	,,	,,	50	208	5600	184496	9″×4″×21
60′-0″	20′-0″	,,	,,	60	300	6720	266100	8″×5″×28
70′-0″	23′-4″	,,	,,	70	408	7840	361896	10″×5″×30
80′-0″	26′-8″	,,	,,	80	533	8960	472771	12″×5″×32
90′-0″	30′-0″	,,	,,	90	675	10080	598725	12″×6″×44
100′-0″	33′-4″	,,	,,	100	833	11200	738871	14″×6″×46
Type 3				**Stancheons 15′-0″ Centres**				
40′-0″	13′-4″	,,	,,	50	167	5600	148129	8″×4″×18
50′-0″	16′-8″	,,	,,	62·5	260	7000	230620	8″×5″×28
60′-0″	20′-0″	,,	,,	75	375	8400	332625	10″×5″×30
70′-0″	23′-4″	,,	,,	87·5	510	9800	452370	12″×5″×32
80′-0″	26′-8″	,,	,,	100	667	11200	591629	12″×6″×44
90′-0″	30′-0″	,,	,,	112·5	844	12600	748628	14″×6″×46
100′-0″	33′-4″	,,	,,	125	1042	14000	924254	14″×6″×57

STANDARD SECTIONS FOR SIDE STANCHEONS.

TYPE 4.

OPEN SHEDS—SHEETED ROOFS.



Type 3.

Span of Shed	Height of Shed	Unit vert. load	Unit B.M.	Multipliers		Total V.L.	Total B.M.	I Section Required
				V.L.	B.M			
Type 4				**Stancheons 12'-0" Centres**				
		lbs.	inch-lbs.			lbs.	inch-lbs.	lbs.
40'-0"	13'-4"	118	444	40	133	4720	59052	7"×4"×16
50'-0"	16'-8"	„	„	50	208	5900	92352	7"×4"×16
60'-0"	20'-0"	„	„	60	300	7080	133200	8"×4"×18
70'-0"	23'-4"	„	„	70	408	8260	181152	9"×4"×21
80'-0"	26'-8"	„	„	80	533	9440	236652	8"×5"×28
90'-0"	30'-0"	„	„	90	675	10620	299700	10"×5"×30
100'-0"	33'-4"	„	„	100	833	11800	369852	10"×5"×30
Type 4				**Stancheons 15'-0" Centres**				
40'-0"	13'-4"	„	„	50	167	5900	74148	7"×4"×16
50'-0"	16'-8"	„	„	62·5	260	7375	115440	7"×4"×16
60'-0"	20'-0"	„	„	75	375	8850	166500	9"×4"×21
70'-0"	23'-4"	„	„	87·5	510	10325	226440	8"×5"×28
80'-0"	26'-8"	„	„	100	667	11800	296148	10"×5"×30
90'-0"	30'-0"	„	„	112·5	844	13275	374736	10"×5"×30
100'-0"	33'-4"	„	„	125	1042	14750	462648	12"×5"×32

Sheet 3/3

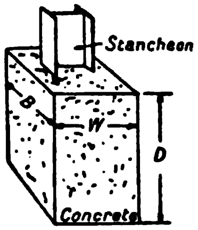
CONCRETE FOUNDATIONS.

OPEN SHEDS—SHEETED ROOFS.

Basis : Earth pressure for Types 2 and 4 : 3 tons per sq. ft.
„ „ „ Type 3 : 1 ton „ „

For Side Stancheons 12′-0″ Centres				
		D W B	B W D	D W B
40′-0″	13′-4″	3′-0″ × 1′-9″ × 1′-9″	2′-0″ × 2′-0″ × 1′-3″	2′-6″ × 1′-9″ × 1′-9″
50′-0″	16′-8″	3′-6″ × 2′-0″ × 2′-0″	2′-0″ × 2′-0″ × 1′-3″	2′-6″ × 1′-9″ × 1′-9″
60′-0″	20′-0″	4′-0″ × 2′-0″ × 2′-0″	2′-1″ × 2′-1″ × 1′-4″	3′-0″ × 2′-0″ × 2′-0″
70′-0″	23′-4″	4′-6″ × 2′-3″ × 2′-3″	2′-2″ × 2′-2″ × 1′-5″	3′-0″ × 2′-0″ × 2′-0″
80′-0″	26′-8″	5′-0″ × 2′-6″ × 2′-6″	2′-3″ × 2′-3″ × 1′-5″	3′-3″ × 2′-0″ × 2′-0″
90′-0″	30′-0″	5′-6″ × 2′-9″ × 2′-9″	2′-4″ × 2′-4″ × 1′-6″	3′-6″ × 2′-3″ × 2′-3″
100′-0″	33′-4″	6′-0″ × 3′-0″ × 3′-0″	2′-6″ × 2′-6″ × 1′-6″	4′-0″ × 2′-6″ × 2′-6″
For Side Stancheons 15′-0″ Centres				
40′-0″	13′-4″	3′-6″ × 2′-0″ × 2′-0″	2′-3″ × 2′-3″ × 1′-3″	2′-6″ × 1′-9″ × 1′-9″
50′-0″	16′-8″	4′-0″ × 2′-0″ × 2′-0″	2′-6″ × 2′-6″ × 1′-4″	2′-6″ × 2′-0″ × 2′-0″
60′-0″	20′-0″	4′-6″ × 2′-3″ × 2′-3″	2′-9″ × 2′-9″ × 1′-4″	2′-9″ × 2′-0″ × 2′-0″
70′-0″	23′-4″	5′-0″ × 2′-6″ × 2′-6″	2′-10″ × 2′-10″ × 1′-5″	3′-0″ × 2′-3″ × 2′-3″
80′-0″	26′-8″	5′-6″ × 2′-9″ × 2′-9″	3′-0″ × 3′-0″ × 1′-6″	3′-3″ × 2′-6″ × 2′-6″
90′-0″	30′-0″	6′-0″ × 3′-0″ × 3′-0″	3′-1″ × 3′-1″ × 1′-8″	3′-9″ × 2′-9″ × 2′-9″
100′-0″	33′-4″	6′-6″ × 3′-3″ × 3′-3″	3′-3″ × 3′-3″ × 1′-9″	4′-3″ × 3′-0″ × 3′-0″

Sheet 3/4



CHAPTER IV.

THE STANDARDIZING OF SHED-MEMBERS AND FOUNDATIONS FOR SHEDS HAVING SLATED ROOFS AND BRICK-PANELLED WALLS.

By adopting a constant outline for the principal members of the roof principals and transverse frames, by using a constant relative proportion for the main dimensions, and also a similar combination of data, the stresses and sections may be expeditiously determined in a similar manner to that described for sheeted sheds.

The data to be adopted for use in the determination of the unit stresses are as follows :—

Weight of roof covering and framework : 18 lbs. per sq. ft.
„ sides (9″ brick walls and steel) : 90 „ „
Roof-wind force : 32 lbs. per sq. ft.—horizontal.
Side „ „ 20 „ „ „
Angle of roof slope : 30°.
Length of building considered : 1′-0″.
Span (transverse distance between side stancheons) : 12′-0″.
Height from ground to eaves : 4′-0″.

$$\text{Unit length} \times \frac{\text{Span}}{12} = \text{total length of member.}$$

VALUES OF SYMBOLS TO SUIT 30° IN LIEU OF 24½° SLOPE ARE :—

B = length of roof slope (for $A = 30°$, $B = \cdot 58C$).
D (for $A = 30°$) $= \cdot 3364C$.
D^1 (for $A = 30°$) $= \cdot 3364C + \cdot 5774S^2$.
VN (for $A = 30°$) $= \cdot 86$ NP.
HN (for $A = 30°$) $= \cdot 5$ NP.

FORCE CONSTANTS FOR REACTIONS.

HP for roof : 32 lbs. per sq. ft. Resolving these, then
NP for roof $= 32 \times \cdot 66 = 21\frac{1}{3}$ lbs. per sq. ft.
VN „ „ $= 21\frac{1}{3} \times \cdot 86 = 18\cdot 35$ lbs. per sq. ft.
HN „ „ $= 21\frac{1}{3} \times \cdot 5 = 10\frac{2}{3}$ lbs. per sq. ft.
Length of roof slope relative to span = 6·96 ft.
Area of roof = 6·96 sq. ft.

Total VN $= 18{\cdot}35 \times 6{\cdot}96 = 127{\cdot}7$ lbs.
,, HN $= 10{\cdot}66 \times 6{\cdot}96 = 74{\cdot}2$,,
,, NP $= 21\tfrac{1}{3} \times 6{\cdot}96 = 148{\cdot}5$,,
HP for side $= 20$ lbs. per sq. ft.
Height of side $= 4'\text{-}0''$.
Area of side $= 4$ sq. ft.
$HP^S = 20 \times 4 = 80$ lbs.
Weight of roof $(W) = 18 \times 6{\cdot}96 \times 2 = 250{\cdot}56$ lbs.
,, each side $(W_1) = 90 \times 4 = 360$ lbs.

CALCULATIONS FOR REACTIONS.

Type 2, Principals.

Total upward reaction at WRP $= \frac{W}{2} + {\cdot}571\ NP = 125{\cdot}28 + 84{\cdot}8 = 210{\cdot}08$ lbs.

Total upward reaction at LRP $= \frac{W}{2} + {\cdot}289\ NP = 125{\cdot}28 + 42{\cdot}9 = 168{\cdot}18$ lbs.

Total horizontal reaction at WRP $= {\cdot}25\ NP = {\cdot}25 \times 148{\cdot}5 = 37{\cdot}125$ lbs.

Total horizontal reaction at LRP $= {\cdot}25\ NP + {\cdot}25\ HP^S = 37{\cdot}125 + 20 = 57{\cdot}125$ lbs.

Type 3, Transverse Frames.

$$\text{Total upward reaction at WRT} = \frac{W}{2} + W_1 + \left(\frac{C - ({\cdot}3364C + {\cdot}577S^2)}{C} \times VN\right) - \frac{HP^S \times L^2}{C} = 125{\cdot}28 + 360 + 60{\cdot}3 - 13{\cdot}33 = 532{\cdot}25 \text{ lbs.}$$

$$\text{Total upward reaction at LRT} = \frac{W}{2} + W_1 + \left(\frac{{\cdot}3364C + {\cdot}577S^2}{C} \times VN\right) + \frac{HP^S \times L^2}{C} = 125{\cdot}28 + 360 + 67{\cdot}5 + 13{\cdot}33 = 566{\cdot}11 \text{ lbs.}$$

$$\text{Total horizontal reaction at WRT} = {\cdot}25\ NP + \frac{HP^S - RHP}{2} + RHP = 37{\cdot}125 + 25 + 30 = 92{\cdot}125 \text{ lbs.}$$

$$\text{Total horizontal reaction at LRT} = {\cdot}25\ NP + \frac{HP^S - RHP}{2} = 37{\cdot}125 + 25 = 62{\cdot}125 \text{ lbs.}$$

Type 4, Transverse Frames.

$$\text{Total upward reaction at WRT} = \frac{W}{2} + W_1 + \left(\frac{C - ({\cdot}3364C + {\cdot}577S^2)}{C} \times VN\right) - \frac{HP^S \times L^2}{C} = 125{\cdot}28 + 360 + 69{\cdot}39 - 5{\cdot}2 = 549{\cdot}47 \text{ lbs.}$$

$$\text{Total upward reaction at LRT} = \frac{W}{2} + W_1 + \left(\frac{{\cdot}3364C + {\cdot}577S^2}{C} \times VN\right) + \frac{HP^S \times L^2}{C} = 125{\cdot}28 + 360 + 58{\cdot}308 + 5{\cdot}2 = 548{\cdot}788 \text{ lbs.}$$

Total horizontal reaction at $\text{WRT} = \cdot 25\,\text{NP} + \dfrac{\text{HP}^{\text{S}} - \text{RHP}}{2} + \text{RHP}$
$= 37\cdot 125 + 17\cdot 5 + 15 = 69\cdot 625$ lbs.

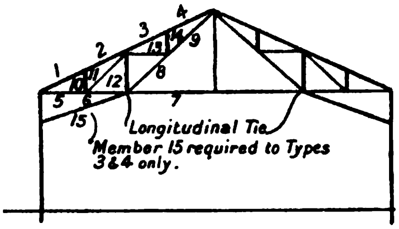
Total horizontal reaction at $\text{LRT} = \cdot 25\,\text{NP} + \dfrac{\text{HP}^{\text{S}} - \text{RHP}}{2}$
$= 37\cdot 125 + 17\cdot 5 = 54\cdot 625$ lbs.

These reactions will enable the force diagrams to be completed, from which the stress diagrams may be drawn in the usual manner, as shown on Plates 4/H, 4/J, and 4/K for Types 2, 3, and 4 respectively, and the amount and nature of the various unit stresses in the members determined. These unit stresses are tabulated on Sheet 4/1 for spans ranging from 40 to 70 ft., and on Sheet 4/2 for spans from 70 ft. to 100 ft., for which, see pages 73 and 74.

By the use of these tables in conjunction with the multipliers referred to in the previous chapter, the total stresses in the various members for sheds of varying spans (but with height at the fixed ratio to the span) may be easily found. Sheets 4/3 to 4/8 give the tabulated stresses for spans varying from 40 to 70 ft., and Sheets 4/9 to 4/14 for spans varying from 70 to 100 ft. It should be noted that the frame diagram for the latter range of spans is a modification of that used for finding the unit stresses, consequently, for certain members, the stresses will require to be separately computed, and although no diagram nor calculations are given, the unit stresses given on Sheet 4/2 for these particular figures may be safely accepted as correct.

SLATED TRUSSES. UNIT STRESS COEFFICIENTS. SPANS 40 TO 70 FT.

SLATED ROOFS.



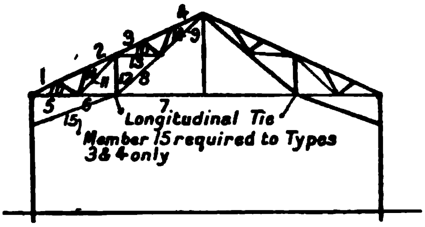
Member	Condition of Ends	Unit Length	Unit stress in pounds		
			Type 2	Type 3	Type 4
1	B.E.F.	1·732	+360	+417	+397
2	B.E.F.	1·732	+378	+435	+415
3	B.E.F.	1·732	+396	+453	+433
4	B.E.F.	1·732	+414	+471	+451
5		1·5	−317	−359	−304
6		1·5	−253	−332	−277
7		3	−125	−112	−126
8	O.E.F.	2·29	−196	−286 or +40	−250
9		2·29	−294	−384	−348
10	O.E.F.	·8661	+74	+74	+74
11		2·29	−98	−98	−98
12	B.E.F.	1·7322	+148	+148	+148
13		1·5	−64	−64	−64
14	O.E.F.	·8661	+74	+74	+74
15	B.E.F.	3·1625		+256	+146

B.E.F. denotes both ends fixed.
O.E.F. denotes one end fixed, other end free.
Unit length multiplied by multiplier equals length of member.
+ denotes compression.
— denotes tension.

Sheet 4/1

TABLE OF STRESS COEFFICIENTS.
SPANS 70 TO 100 FT.

SLATED ROOFS.



Member	Condition of Ends	Unit Length	Table of stress coefficients		
			Type 2	Type 3	Type 4
1	B.E.F.	1·155	+378	+440	+409
2	B.E.F.	1·155	+398	+460	+429
3	B.E.F.	1·155	+420	+482	+451
4	B.E.F.	1·155	+440	+502	+471
5		1·5	−340	−389	−320
6		1·5	−253	−352	−283
7		3	−125	−112	−123
8	O.E.F.	2·29	−179	−283 or +23	−236 or +16
9		2·29	−317	−421	−374
10	O.E.F.	·764	+44	+44	+44
11		2·29	−138	−138	−138
12	B.E.F.	1·7322	+147	+147	+147
13		1·5	−87	−87	−87
14	O.E.F.	1·258	+69	+69	+69
15	B.E.F.	3·1625		+256	+146

B.E.F. denotes both ends fixed.

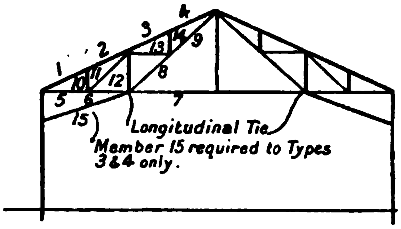
SHEET 4/2

O.E.F. denotes one end fixed, other end free.
Unit length multiplied by multiplier equals length of member.
+ denotes compression.
− denotes tension.

TOTAL STRESSES IN SPANS 40 TO 70 FT.

12′-0″ CENTRES—TYPE 2.

SLATED ROOFS.



Member	Condition of Ends	Unit Length	40′-0″ Span	50′-0″ Span	60′-0″ Span	70′-0″ Span
			pounds	pounds	pounds	pounds
1	B.E.F.	1·732	+14400	+18000	+21600	+25200
2	B.E.F.	1·732	+15120	+18900	+22680	+26460
3	B.E.F.	1·732	+15840	+19800	+23760	+27720
4	B.E.F.	1·732	+16560	+20700	+24840	+28980
5		1·5	−12680	−15850	−19020	−22190
6		1·5	−10120	−12650	−15180	−17710
7		3	− 5000	− 6250	− 7500	− 8750
8		2·29	− 7840	− 9800	−11760	−13720
9		2·29	−11760	−14700	−17640	−20580
10	O.E.F.	·8661	+ 2960	+ 3700	+ 4440	+ 5180
11		2·29	− 3920	− 4900	− 5880	− 6860
12	B.E.F.	1·7322	+ 5920	+ 7400	+ 8880	+10360
13		1·5	− 2560	− 3200	− 3840	− 4480
14	O.E.F.	·8661	+ 2960	+ 3700	+ 4440	+ 5180

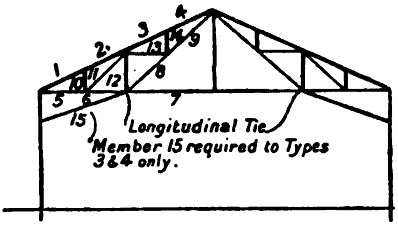
SHEET 4/3

B.E.F. denotes both ends fixed.
O.E.F. denotes one end fixed, other end free.
Unit length multiplied by multiplier equals length of member.
+ denotes compression
— denotes tension.

TOTAL STRESSES IN SPANS 40 TO 70 FT.

15′-0″ CENTRES—TYPE 2.

SLATED ROOFS.



Member	Condition of Ends	Unit Length	40′-0″ Span	50′-0″ Span	60′-0″ Span	70′-0″ Span
			pounds	pounds	pounds	pounds
1	B.E.F.	1·732	+18000	+22500	+27000	+31500
2	B.E.F.	1·732	+18900	+23625	+28350	+33075
3	B.E.F.	1·732	+19800	+24750	+29700	+34650
4	B.E.F.	1·732	+20700	+25875	+31050	+36225
5		1·5	−15850	−19813	−23775	−27738
6		1·5	−12650	−15813	−18975	−22138
7		3	− 6250	− 7813	− 9375	−10938
8		2·29	− 9800	−12250	−14700	−17150
9		2·29	−14700	−18375	−22050	−25725
10	O.E.F.	·8661	+ 3700	+ 4625	+ 5550	+ 6475
11		2·29	− 4900	− 6125	− 7350	− 8575
12	B.E.F.	1·7322	+ 7400	+ 9250	+11100	+12950
13		1·5	− 3200	− 4000	− 4800	− 5600
14	O.E.F.	·8661	+ 3700	+ 4625	+ 5550	+ 6475

B.E.F denotes both ends fixed.

SHEET 4/4

O.E.F. denotes one end fixed, other end free.

Unit length multiplied by multiplier equals length of member.

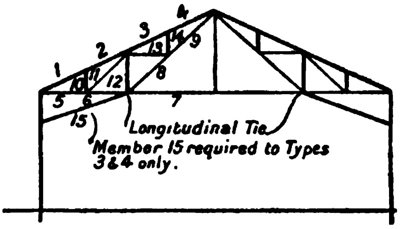
+ denotes compression.

— denotes tension.

TOTAL STRESSES IN SPANS 40 TO 70 FT.

12′-0″ CENTRES—TYPE 3.

SLATED ROOFS.



Member	Condition of Ends	Unit Length	40′-0″ Span	50′-0″ Span	60′-0″ Span	70′-0″ Span
			pounds	pounds	pounds	pounds
1	B.E.F.	1·732	+16680	+20850	+25020	+29190
2	B.E.F.	1·732	+17400	+21750	+26100	+30450
3	B.E.F.	1·732	+18120	+22650	+27180	+31710
4	B.E.F.	1·732	+18840	+23550	+28260	+32970
5		1·5	−14360	−17950	−21540	−25130
6		1·5	−13280	−16600	−19920	−23240
7		3	− 4480	− 5600	− 6720	− 7840
8	O.E.F.	2·29	−11440 or + 1600	−14300 or + 2000	−17160 or + 2400	−20020 or + 2800
9		2·29	−15360	−19200	−23040	−26880
10	O.E.F.	·8661	+ 2960	+ 3700	+ 4440	+ 5180
11		2·29	− 3920	− 4900	− 5880	− 6860
12	B.E.F.	1·7322	+ 5920	+ 7400	+ 8880	+10360
13		1·5	− 2560	− 3200	− 3840	− 4480
14	O.E.F.	·8661	+ 2960	+ 3700	+ 4440	+ 5180
15	B.E.F.	3·1625	+10240	+12800	+15360	+17920

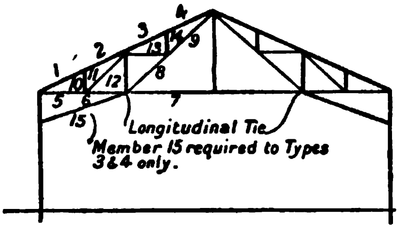
B.E.F. denotes both ends fixed.
O.E.F. denotes one end fixed, other end free.
Unit length multiplied by multiplier equals length of member.
+ denotes compression.
− denotes tension.

SHEET 4/5

TOTAL STRESSES IN SPANS 40 TO 70 FT.

15′-0″ CENTRES—TYPE 3.

SLATED ROOFS.



Member	Condition of Ends	Unit Length	40′-0″ Span	50′-0″ Span	60′-0″ Span	70′-0″ Span
			pounds	pounds	pounds	pounds
1	B.E.F.	1·732	+20850	+26063	+31275	+36488
2	B.E.F.	1·732	+21750	+27188	+32625	+38063
3	B.E.F.	1·732	+22650	+28313	+33975	+39638
4	B.E.F.	1·732	+23550	+29438	+35325	+41213
5		1·5	−17950	−22438	−26925	−31413
6		1·5	−16600	−20750	−24900	−29050
7		3	− 5600	− 7000	− 8400	− 9800
8	O.E.F.	2·29	−14300 or + 2000	−17875 or + 2500	−21450 or + 3000	−25025 or + 3500
9		2·29	−18200	−24000	−28800	−33600
10	O.E.F.	·8661	+ 3700	+ 4625	+ 5550	+ 6475
11		2·29	− 4900	− 6125	− 7350	− 8575
12	B.E.F.	1·7322	+ 7400	+ 9250	+11100	+12950
13		1·5	− 3200	− 4000	− 4800	− 5600
14	O.E.F.	·8661	+ 3700	+ 4625	+ 5550	+ 6475
15	B.E.F.	3·1625	+12800	+16000	+19200	+22400

B.E.F. denotes both ends fixed.

SHEET 4/6

O.E.F. denotes one end fixed, other end free.

Unit length multiplied by multiplier equals length of member.

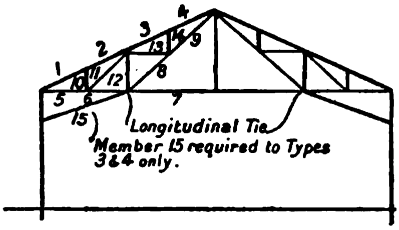
+ denotes compression.

− denotes tension.

TOTAL STRESSES IN SPANS 40 TO 70 FT.

12′-0″ CENTRES—TYPE 4.

SLATED ROOFS.



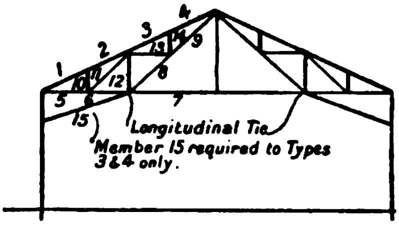
Member	Condition of Ends	Unit Length	40′-0″ Span	50′-0″ Span	60′-0″ Span	70′-0″ Span
			pounds	pounds	pounds	pounds
1	B.E.F.	1·732	+15880	+19850	+23820	+27790
2	B.E.F.	1·732	+16600	+20750	+24900	+29050
3	B.E.F.	1·732	+17320	+21650	+25980	+30310
4	B.E.F.	1·732	+18040	+22550	+27060	+31570
5		1·5	−12160	−15200	−18240	−21280
6		1·5	−11080	−13850	−16620	−19390
7		3	− 5040	− 6300	− 7560	− 8820
8		2·29	−10000	−12500	−15000	−17500
9		2·29	−13920	−17400	−20880	−24360
10	O.E.F.	·8661	+ 2960	+ 3700	+ 4440	+ 5180
11		2·29	− 3920	− 4900	− 5880	− 6860
12	B.E.F.	1·7322	+ 5920	+ 7400	+ 8880	+10360
13		1·5	− 2560	− 3200	− 3840	− 4480
14	O.E.F.	·8661	+ 2960	+ 3700	+ 4440	+ 5180
15	B.E.F.	3·1625	+ 5840	+ 7300	+ 8760	+10220

B.E.F. denotes both ends fixed. SHEET 4/7
O.E.F. denotes one end fixed, other end free.
Unit length multiplied by multiplier equals length of member.
+ denotes compression.
− denotes tension.

TOTAL STRESSES IN SPANS 40 TO 70 FT.

15′-0″ CENTRES—TYPE 4.

SLATED ROOFS.



Member	Condition of Ends	Unit Length	40′-0″ Span	50′-0″ Span	60′-0″ Span	70′-0″ Span
			pounds	pounds	pounds	pounds
1	B.E.F.	1·732	+19850	+24813	+29775	+34738
2	B.E.F.	1·732	+20750	+25938	+31125	+36313
3	B.E.F.	1·732	+21650	+27063	+32475	+37888
4	B.E.F.	1·732	+22550	+28188	+33825	+39463
5		1·5	−15200	−19000	−22800	−26600
6		1·5	−13850	−17313	−20775	−24238
7		3	− 6300	− 7875	− 9450	−11025
8		2·29	−12500	−15625	−18750	−21875
9		2·29	−17400	−21750	−26100	−30450
10	O.E.F.	·8661	+ 3700	+ 4625	+ 5550	+ 6475
11		2·29	− 4900	− 6125	− 7350	− 8575
12	B.E.F.	1·7322	+ 7400	+ 9250	+11100	+12950
13		1·5	− 3200	− 4000	− 4800	− 5600
14	O.E.F.	·8661	+ 3700	+ 4625	+ 5550	+ 6475
15	B.E.F.	3·1625	+ 7300	+ 9125	+10950	+12775

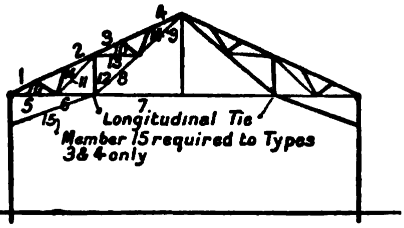
B.E.F. denotes both ends fixed.
O.E.F. denotes one end fixed, other end free.
Unit length multiplied by multiplier equals length of member.
+ denotes compression.
— denotes tension.

Sheet 4/8

TOTAL STRESSES IN SPANS 70 TO 100 FT.

12′-0″ CENTRES—TYPE 2.

SLATED ROOFS.



Member	Condition of Ends	Unit Length	70′-0″ Span	80′-0″ Span	90′-0″ Span	100′-0″ Span
			pounds	pounds	pounds	pounds
1	B.E.F.	1·155	+26460	+30240	+34020	+37800
2	B.E.F.	1·155	+27860	+31840	+35820	+39800
3	B.E.F.	1·155	+29400	+33600	+37800	+42000
4	B.E.F.	1·155	+30800	+35200	+39600	+44000
5		1·5	−23800	−27200	−30600	−34000
6		1·5	−17710	−20240	−22770	−25300
7		3	− 8750	−10000	−11250	−12500
8		2·29	−12530	−14320	−16110	−17900
9		2·29	−22190	−25360	−28530	−31700
10	O.E.F.	·764	+ 3080	+ 3520	+ 3960	+ 4400
11		2·29	− 9660	−11040	−12420	−13800
12	B.E.F.	1·7322	+10290	+11760	+13230	+14700
13		1·5	− 6090	− 6960	− 7830	− 8700
14	O.E.F.	1·258	+ 4830	+ 5520	+ 6210	+ 6900

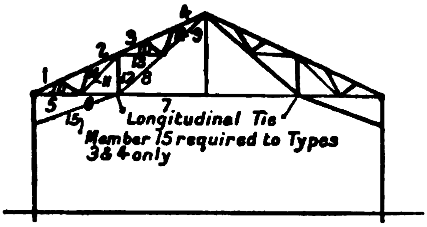
B.E.F. denotes both ends fixed.
O.E.F. denotes one end fixed, other end free.
Unit length multiplied by multiplier equals length of member.
+ denotes compression.
— denotes tension.

Sheet 4/9

TOTAL STRESSES IN SPANS 70 TO 100 FT.

15′-0″ CENTRES—TYPE 2.

SLATED ROOFS.



Member	Condition of Ends	Unit Length	70′-0″ Span	80′-0″ Span	90′-0″ Span	100′-0″ Span
			pounds	pounds	pounds	pounds
1	B.E.F.	1·155	+33075	+37800	+42525	+47250
2	B.E.F.	1·155	+34825	+39800	+44775	+49750
3	B.E.F.	1·155	+36750	+42000	+47250	+52500
4	B.E.F.	1·155	+38500	+44000	+49500	+55000
5		1·5	−29750	−34000	−38250	−42500
6		1·5	−22138	−25300	−28463	−31625
7		3	−10938	−12500	−14063	−15625
8		2·29	−15663	−17900	−20138	−22375
9		2·29	−27738	−31700	−35663	−39625
10	O.E.F.	·764	+ 3850	+ 4400	+ 4950	+ 5500
11		2·29	−12075	−13800	−15525	−17250
12	B.E.F.	1·7322	+12863	+14700	+16538	+18375
13		1·5	− 7613	− 8700	− 9788	−10875
14	O.E.F.	1·258	+ 6038	+ 6900	+ 7763	+ 8625

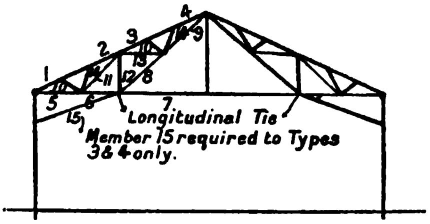
B.E.F. denotes both ends fixed.
O.E.F denotes one end fixed, other end free.
Unit length multiplied by multiplier equals length of member.
+ denotes compression.
— denotes tension.

SHEET 4/10

TOTAL STRESSES IN SPANS 70 TO 100 FT.

12'-0" CENTRES—TYPE 3:

SLATED ROOFS.



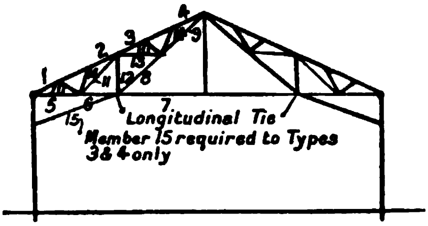
Member	Condition of Ends	Unit Length	70'-0" Span	80'-0" Span	90'-0" Span	100'-0" Span
			pounds	pounds	pounds	pounds
1	B.E.F.	1·155	+30800	+35200	+39600	+44000
2	B.E.F.	1·155	+32200	+36800	+41400	+46000
3	B.E.F.	1·155	+33740	+38560	+43380	+48200
4	B.E.F.	1·155	+35140	+40160	+45180	+50200
5		1·5	−27230	−31120	−35010	−38900
6		1·5	−24640	−28160	−31680	−35200
7		3	− 7840	− 8960	−10080	−11200
8	O.E.F.	2·29	+1610 or −19810	+1840 or −22640	+ 2070 or −25470	+2300 or −28300
9		2·29	−29470	−33680	−37890	−42100
10	O.E.F.	·764	+ 3080	+ 3520	+ 3960	+ 4400
11		2·29	− 9660	−11040	−12420	−13800
12	B.E.F.	1·7322	+10290	+11760	+13230	+14700
13		1·5	− 6090	− 6960	− 7830	− 8700
14	O.E.F.	1·258	+ 4830	+ 5520	+ 6210	+ 6900
15	B.E.F.	3·1625	+17920	+20480	+23040	+25600

B.E.F. denotes both ends fixed. SHEET 4/11
O.E.F. denotes one end fixed, other end free.
Unit length multiplied by multiplier equals length of member.
+ denotes compression.
— denotes tension.

TOTAL STRESSES IN SPANS 70 TO 100 FT.

15′-0″ CENTRES—TYPE 3.

SLATED ROOFS.



Member	Condition of Ends	Unit Length	70′-0″ Span	80′-0″ Span	90′-0″ Span	100′-0″ Span
			pounds	pounds	pounds	pounds
1	B.E.F.	1·155	+38500	+44000	+49500	+55000
2	B.E.F.	1·155	+40250	+46000	+51750	+57500
3	B.E.F.	1·155	+42175	+48200	+54225	+60250
4	B.E.F.	1·155	+43925	+50200	+56475	+62750
5		1·5	−34038	−38900	−43763	−48625
6		1·5	−30800	−35200	−39600	−44000
7		3	− 9800	−11200	−12600	−14000
8	O.E.F.	2·29	+ 2013 or −24763	+ 2300 or −28300	+ 2588 or −31838	+ 2785 or −35375
9		2·29	−36838	−42100	−47363	−52625
10	O.E.F.	·764	+ 3850	+ 4400	+ 4950	+ 5500
11		2·29	−12075	−13800	−15525	−17250
12	B.E.F.	1·7322	+12863	+14700	+16538	+18375
13		1·5	− 7613	− 8700	− 9788	−10875
14	O.E.F.	1·258	+ 6038	+ 6900	+ 7763	+ 8625
15	B.E.F.	3·1625	+22400	+25600	+28800	+32000

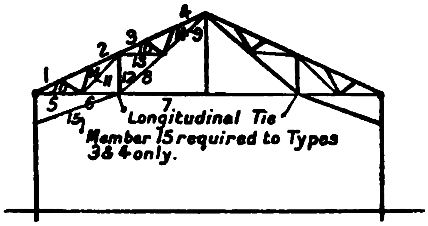
B.E.F. denotes both ends fixed.
O.E.F. denotes one end fixed, other end free.
Unit length multiplied by multiplier equals length of member.
+ denotes compression.
— denotes tension.

SHEET 4/12

TOTAL STRESSES IN SPANS 70 TO 100 FT.

12′-0″ CENTRES—TYPE 4.

SLATED ROOFS.



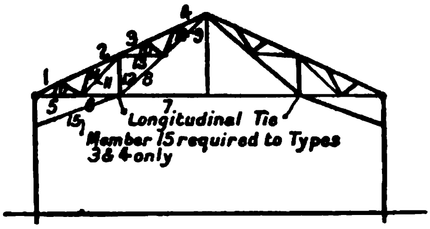
Member	Condition of Ends	Unit Length	70′-0″ Span	80′-0″ Span	90′-0″ Span	100′-0″ Span
			pounds	pounds	pounds	pounds
1	B.E.F.	1·155	+28630	+32720	+36810	+40900
2	B.E.F.	1·155	+30030	+34320	+38610	+42900
3	B.E.F.	1·155	+31570	+36080	+40590	+45100
4	B.E.F.	1·155	+32970	+37680	+42390	+47100
5		1·5	−22400	−25600	−28800	−32000
6		1·5	−19810	−22640	−25470	−28300
7		3	− 8610	− 9840	−11070	−12300
8	O.E.F.	2·29	+ 1120 or −16520	+ 1280 or −18880	+ 1440 or −21240	+ 1600 or −23600
9		2·29	−26180	−29920	−33660	−37400
10	O.E.F.	·764	+ 3080	+ 3520	+ 3960	+ 4400
11		2·29	− 9660	−11040	−12420	−13800
12	B.E.F.	1·7322	+10290	+11760	+13230	+14700
13		1·5	− 6090	− 6960	− 7830	− 8700
14	O.E.F.	1·258	+ 4830	+ 5520	+ 6210	+ 6900
15	B.E.F.	3·1625	+10220	+11680	+13140	+14600

B.E.F. denotes both ends fixed. SHEET 4/13
O.E.F. denotes one end fixed, other end free.
Unit length multiplied by multiplier equals length of member.
+ denotes compression.
— denotes tension.

TOTAL STRESSES IN SPANS 70 TO 100 FT.

15′-0″ CENTRES—TYPE 4.

SLATED ROOFS.



Member	Condition of Ends	Unit Length	70′-0″ Span	80′-0″ Span	90′-0″ Span	100′-0″ Span
			pounds	pounds	pounds	pounds
1	B.E.F.	1·155	+35788	+40900	+46013	+51125
2	B.E.F.	1·155	+37538	+42900	+48263	+53625
3	B.E.F.	1·155	+39463	+45100	+50738	+56375
4	B.E.F.	1·155	+41213	+47100	+52988	+58785
5		1·5	−28000	−32000	−36000	−40000
6		1·5	−24763	−28300	−31838	−35375
7		3	−10763	−12300	−13838	−15375
8	O.E.F.	2·29	+ 1400 or −20650	+ 1600 or −23600	+ 1800 or −26550	+ 2000 or −29500
9		2·29	−32725	−37400	−42075	−46750
10	O.E.F.	·764	+ 3850	+ 4400	+ 4950	+ 5500
11		2·29	−12075	−13800	−15525	−17250
12	B.E.F.	1·7322	+12863	+14700	+16538	+18375
13		1·5	− 7613	− 8700	− 9788	−10875
14	O.E.F.	1·258	+ 6038	+ 6900	+ 7763	+ 8625
15	B.E.F.	3·1625	+12775	+14600	+16425	+18250

B.E.F. denotes both ends fixed.

SHEET 4/14

O.E.F. denotes one end fixed, other end free.
Unit length multiplied by multiplier equals length of member.
+ denotes compression.
— denotes tension.

STANDARD SECTIONS FOR PRINCIPALS AND TRANSVERSE FRAMES.

The sections recommended for the members of the principals and for the stays are given on Sheets 4/15 to 4/18, and have been calculated to resist the tabulated total stresses, the allowable safe stress per square inch being as per previous chapter.

The method previously described for increasing the utility of the standard sections may also be applied here.

SECTIONS FOR 70′-0″ TO 100′-0″ SPAN GABLE FRAMES AND BRACINGS TO SUIT PRINCIPALS 12′-0″ CENTRES.

For outline diagram see Sheet 4/26.

SLATED ROOFS.

Member		70′-0″ Span	80′-0″ Span	90′-0″ Span	100′-0″ Span
Upright	U	10″×5″×30 lbs. I	12″×5″×32 lbs. I	14″×6″×46 lbs. I	14″×6″×57 lbs. I
„	U^1	7″×4″×16 lbs. I	7″×4″×16 lbs. I	8″×4″×18 lbs. I	9″×4″×21 lbs. I
„	U^2	9″×4″×21 lbs. I	9″×4″×21 lbs. I	10″×5″×30 lbs. I	10″×5″×30 lbs. I
Rail	R^2	12″×3″ ⊓	12″×3″ ⊓	12″×3″ ⊓	12″×3″ ⊓
Rafter	R	12″×3″ ⊓	12″×3″ ⊓	12″×3″ ⊓	12″×3″ ⊓
Strut	S	2/3″×2″×½″ ┐┌	2/3″×2″×½″ ┐┌	2/4″×2″×½″ ┐┌	2/4″×2″×½″ ┐┌
„	S^1	2/3″×2″×½″ ┐┌	2/3″×2″×½″ ┐┌	2/4″×2″×½″ ┐┌	2/4″×2″×½″ ┐┌
Tie	B^1	1/3″×3″×½″ L	1/3″×3″×⁵⁄₁₆″ L	1/4″×3″×⁵⁄₁₆″ L	1/5″×3″×⁵⁄₁₆″ L
„	B^2	1/3″×3″×½″ L	1/3″×3″×⁵⁄₁₆″ L	1/4″×3″×⁵⁄₁₆″ L	1/5″×3″×⁵⁄₁₆″ L
„	T^1	1/4″×3″×⁵⁄₁₆″ L	1/4″ ×3″ ×⁵⁄₁₆″ L	1/4½″×3″ ×⁵⁄₁₆″ L	1/5″ ×3″×⁵⁄₁₆″ L
Strut	S^5	2/5″×3″×⁵⁄₁₆″ ┐┌	2/5″ ×3″ ×⅜″ ┐┌	2/5″ ×3½″×⅜″ ┐┌	2/6″ ×4″×⅜″ ┐┌
„	S^6	2/3″×2″×½″ ┐┌	2/3″ ×2″ ×½″ ┐┌	2/3″ × 2″×½″ ┐┌	2/3½″×2″×½″ ┐┌
„	S^7	2/3″×2½″×⁵⁄₁₆″ ┐┌	2/3½″×3″ ×⁵⁄₁₆″ ┐┌	2/4″ ×3″ ×⁵⁄₁₆″ ┐┌	2/5″ ×3″×⁵⁄₁₆″ ┐┌
Flange	F	1/12″×3″ ⊓	1/12″×3″ ⊓	1/12″×3½″ ⊓	1/12″×4″ ⊓
„	F^1	2/3″×2½″×⁵⁄₁₆″ ┐┌	2/3½″×2½″×⁵⁄₁₆″ ┐┌	2/4″ ×3″ ×⁵⁄₁₆″ ┐┌	2/5″ ×3″ ×⅜″ ┐┌

Sheet 4/24

Note.—Uprights U, U^1, and U^2 will suit a 9″ wall best if the depth is not less than 12″.

SECTIONS FOR 70′-0″ TO 100′-0″ SPAN GABLE FRAMES AND BRACINGS TO SUIT PRINCIPALS 15′-0″ CENTRES.

For outline diagram see Sheet 4/26.

SLATED ROOFS.

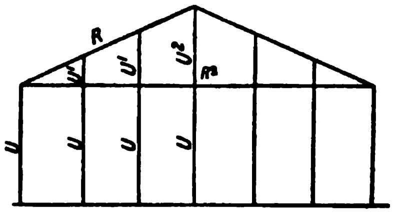
Member	70′-0″ Span	80′-0″ Span	90′-0″ Span	100′-0″ Span
Upright U	10″×5″×30 lbs. I	12″×5″×32 lbs. I	14″×6″×46 lbs. I	14″×6″×57 lbs. I
„ U^1	7″×4″×16 lbs. I	7″×4″×16 lbs. I	8″×4″×18 lbs. I	9″×4″×21 lbs. I
„ U^2	9″×4″×21 lbs. I	9″×4″×21 lbs. I	10″×5″×30 lbs. I	10″×5″×30 lbs. I
Rail R^2	12″×3″ ⊓	12″×3″ ⊓	12″×3½″ ⊓	12″×3½″ ⊓
Rafter R	12″×3″ ⊓	12″×3″ ⊓	12″×3½″ ⊓	12″×3½″ ⊓
Strut S	2/3″×2½″×¼″ ┐┌	2/3½″×2½″×¼″ ┐┌	2/4″×3″×$\frac{5}{16}$″ ┐┌	2/4″×3″×$\frac{5}{16}$″ ┐┌
„ S^1	2/3″×2½″×¼″ ┐┌	2/3½″×2½″×¼″ ┐┌	2/4″×3″×$\frac{5}{16}$″ ┐┌	2/4″×3″×$\frac{5}{16}$″ ┐┌
Tie B^1	1/3″×3″×¼″ └	1/3″×3″ ×$\frac{5}{16}$″ └	1/4″×3″×$\frac{5}{16}$″ └	1/5″×3″×$\frac{5}{16}$″ └
„ B^2	1/3″×3″×¼″ └	1/3″×3″ ×$\frac{5}{16}$″ └	1/4″×3″×$\frac{5}{16}$″ └	1/5″×3″×$\frac{5}{16}$″ └
„ T^1	1/4″×3″×$\frac{5}{16}$″ └	1/4″×3″ ×$\frac{5}{16}$″ └	1/4″×3½″×$\frac{5}{16}$″ └	1/5″×3″×$\frac{5}{16}$″ └
Strut S^5	2/6″×3″×⅜″ ┐┌	2/6″×3½″×⅜″ ┐┌	2/6″×3½″×$\frac{7}{16}$″ ┐┌	2/6″×4″×$\frac{7}{16}$″ ┐┌
„ S^6	2/3″×2½″×¼″ ┐┌	2/3″×2½″×¼″ ┐┌	2/3″×2½″×¼″ ┐┌	2/3½″×2½″×¼″ ┐┌
„ S^7	2/4″×3″×$\frac{5}{16}$″ ┐┌	2/4″×3″×$\frac{5}{16}$″ ┐┌	2/5″×3″×$\frac{5}{16}$″ ┐┌	2/6″×3″×$\frac{5}{16}$″ ┐┌
Flange F	1/12″×3″ ⊓	1/12″×3″ ⊓	1/12″×3½″ ⊓	1/12″×4″ ⊓
„ F^1	2/3½″×2½″×$\frac{5}{16}$″ ┐┌	2/3½″×2½″×$\frac{5}{16}$″ ┐┌	2/4″×3″×$\frac{5}{16}$″ ┐┌	2/5″×3″×⅜″ ┐┌

Sheet 4/25

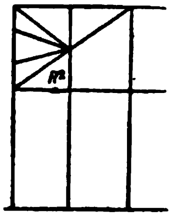
Note.—Uprights U, U^1, and U^2 will suit a 9″ wall best if the depth is not less than 12″.

OUTLINE DIAGRAMS FOR GABLE FRAMES FOR SPANS 70′-0″ TO 100′-0″.

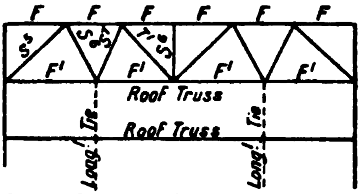
SLATED ROOFS.



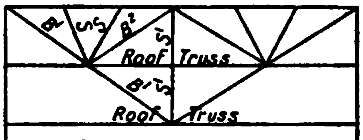
Front Elevation.



Side Elevation.



Plan of Bracing in Plane of Bottom Chord.



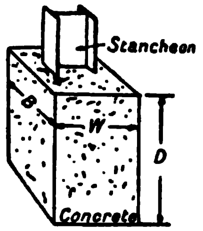
Plan of Bracing in Plane of Roof.

CONCRETE FOUNDATIONS. SHEDS WITH SLATED ROOFS AND BRICK-PANELLED WALLS.

Basis : Earth pressure for Types 2 and 4 : 3 tons per sq. ft.
„ „ „ Type 3 : 1 ton „ „

For Side Stancheons 12′-0″ Centres				
Span	Height	Type 2	Type 3	Type 4
		D W B	B W D	D W B
40′-0″	13′-4″	3′-6″ × 2′-9″ × 2′-9″	2′-6″ × 2′-6″ × 1′-3″	2′-6″ × 2′-9″ × 2′-9″
50′-0″	16′-8″	4′-0″ × 3′-0″ × 3′-0″	2′-10″ × 2′-10″ × 1′-4″	3′-0″ × 3′-0″ × 3′-0″
60′-0″	20′-0″	4′-6″ × 3′-4″ × 3′-4″	3′-2″ × 3′-2″ × 1′-6″	3′-4″ × 3′-4″ × 3′-4″
70′-0″	23′-4″	5′-0″ × 3′-8″ × 3′-8″	3′-6″ × 3′-6″ × 1′-6″	3′-8″ × 3′-8″ × 3′-8″
80′-0″	26′-8″	5′-6″ × 4′-0″ × 4′-0″	3′-10″ × 3′-10″ × 1′-8″	4′-0″ × 4′-0″ × 4′-0″
90′-0″	30′-0″	6′-0″ × 4′-3″ × 4′-3″	4′-2″ × 4′-2″ × 1′-10″	4′-3″ × 4′-3″ × 4′-3″
100′-0″	33′-4″	6′-6″ × 4′-6″ × 4′-6″	4′-6″ × 4′-6″ × 2′-0″	4′-6″ × 4′-6″ × 4′-6″
For Side Stancheons 15′-0″ Centres				
40′-0″	13′-4″	3′-6″ × 3′-3″ × 3′-3″	2′-9″ × 2′-9″ × 1′-3″	2′-9″ × 3′-3″ × 3′-3″
50′-0″	16′-8″	4′-0″ × 3′-8″ × 3′-8″	3′-2″ × 3′-2″ × 1′-4″	3′-0″ × 3′-6″ × 3′-6″
60′-0″	20′-0″	4′-6″ × 4′-0″ × 4′-0″	3′-6″ × 3′-6″ × 1′-6″	3′-4″ × 3′-10″ × 3′-10″
70′-0″	23′-4″	5′-0″ × 4′-4″ × 4′-4″	3′-10″ × 3′-10″ × 1′-6″	3′-8″ × 4′-2″ × 4′-2″
80′-0″	26′-8″	5′-6″ × 4′-9″ × 4′-9″	4′-3″ × 4′-3″ × 1′-8″	4′-0″ × 4′-6″ × 4′-6″
90′-0″	30′-0″	6′-0″ × 5′-2″ × 5′-2″	4′-9″ × 4′-9″ × 1′-10″	4′-3″ × 4′-9″ × 4′-9″
100′-0″	33′-4″	6′-6″ × 5′-6″ × 5′-6″	5′-0″ × 5′-0″ × 2′-0″	4′-6″ × 5′-0″ × 5′-0″

Gable Stancheons		
		W B D
40′-0″	13′-4″	1′-6″ × 1′-6″ × 1′-3″
50′-0″	16′-8″	1′-9″ × 1′-9″ × 1′-6″
60′-0″	20′-0″	2′-0″ × 2′-0″ × 1′-6″
70′-0″	23′-4″	2′-0″ × 2′-0″ × 1′-6″
80′-0″	26′-8″	2′-0″ × 2′-0″ × 1′-6″
90′-0″	30′-0″	2′-3″ × 2′-3″ × 1′-9″
100′-0″	33′-4″	2′-3″ × 2′-3″ × 2′-0″



STANDARD SECTIONS FOR SIDE STANCHEONS.

Type 2.—The maximum bending moment occurs at the ground-level, the unit amount—ignoring effect of wall—being found as follows :—

$$\left.\begin{array}{l}\text{Vertical load } W_2 = 210{\cdot}08 \text{ lbs.} \\ \quad ,, \quad ,, \quad W_1 = 360 \text{ lbs.}\end{array}\right\} 570{\cdot}08 \text{ lbs. (total).}$$

$HN = 37{\cdot}125$ lbs.

Leverage of $HN = 4'\text{-}0''$.

Proportion of $HP^S = 20$ lbs.

Leverage of $HP^S = 4'\text{-}0''$.

$BM = (HN + HP^S) \times 4'\text{-}0'' = (37{\cdot}125 + 20) \times 48'' = 2742$ inch-lbs.

Type 3.—The maximum bending moment occurs at the connection of the stay to the stancheon, the unit amount—ignoring effect of wall—being found as follows :—

Vertical load $= 532{\cdot}26$ lbs.
Horizontal reaction at foot $= 62{\cdot}125$ lbs.
Leverage of do. $= 36''$.
$BM = 62{\cdot}125 \times 36'' = 2236{\cdot}5$ inch-lbs.

Type 4.—The maximum bending moment will be assumed as situated at the ground-level, and the unit amount—ignoring effect of wall—may be found as follows :—

Vertical load $= 549{\cdot}47$ lbs.
Horizontal reaction at point of contraflexure $= 54{\cdot}625$ lbs.
Leverage of do. $= 1'\text{-}6''$.
Side wind on portion below point of contraflexure $= 1\frac{1}{2} \times 20 = 30$ lbs.
Leverage of do. $= 9''$.
$BM = (54{\cdot}625 \times 18'') + (30 \times 9'') = 1253{\cdot}25$ inch-lbs.

The resistance to overturning of the wall itself, being of small moment, has been ignored in the foregoing calculations.

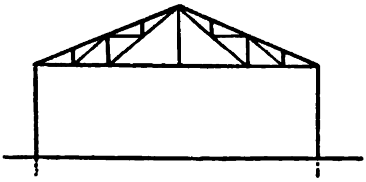
The total vertical loads and also the total bending moments may be found by the formulæ described in the foregoing chapter. These loads and bending moments together with the sections recommended are tabulated hereafter on Sheets 4/19, 4/20, and 4/21, the sections being subjected to the same stress per sq. inch as before for Chapter II.

The method previously described for increasing the utility of the standard sections may also be applied here.

STANDARD SECTIONS FOR SIDE STANCHEONS.

TYPE 2.

SLATED ROOFS.



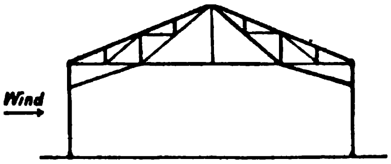
Span of Shed	Height of Shed	Unit vert. load	Unit B.M.	Multipliers		Total V.L.	Total B.M.	I Section Required
				V.L.	B.M.			
Type 2						**Stancheons 12′-0″ Centres**		
		lbs.	inch-lbs.			lbs.	inch-lbs.	lbs.
40′-0″	13′-4″	570	2472	40	133	22800	364686	10″ × 5″ × 30
50′-0″	16′-8″	,,	,,	50	208	28500	570336	12″ × 5″ × 39
60′-0″	20′-0″	,,	,,	60	300	34200	822600	14″ × 6″ × 46
70′-0″	23′-4″	,,	,,	70	408	39900	1118736	14″ × 6″ × 57
80′-0″	26′-8″	,,	,,	80	533	45600	1461486	18″ × 7″ × 75
90′-0″	30′-0″	,,	,,	90	675	51300	1850850	18″ × 7″ × 75
100′-0″	33′-4″	,,	,,	100	833	57000	2284086	20″ × 7½″ × 89
Type 2						**Stancheons 15′-0″ Centres**		
40′-0″	13′-4″	,,	,,	50	167	28500	438720	10″ × 5″ × 30
50′-0″	16′-8″	,,	,,	62·5	260	35625	712920	12″ × 5″ × 39
60′-0″	20′-0″	,,	,,	75	375	42750	1028250	14″ × 6″ × 57
70′-0″	23′-4″	,,	,,	87·5	510	49875	1398420	16″ × 6″ × 62
80′-0″	26′-8″	,,	,,	100	667	57000	1828914	18″ × 7″ × 75
90′-0″	30′-0″	,,	,,	112·5	844	64125	2314248	20″ × 7½″ × 89
100′-0″	33′-4″	,,	,,	125	1042	71250	2857164	24″ × 7½″ × 100

Sheet 4/19

STANDARD SECTIONS FOR SIDE STANCHEONS.

TYPE 3.

SLATED ROOFS.



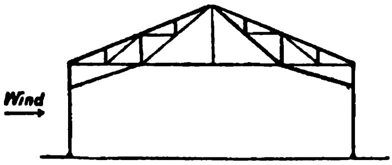
Span of Shed	Height of Shed	Unit vert. load	Unit B.M.	Multipliers		Total V.L.	Total B.M.	I Section Required
				V.L.	B.M.			
Type 3				**Stancheons 12′-0″ Centres**				
		lbs.	inch-lbs.			lbs.	inch-lbs.	lbs.
40′-0″	13′-4″	533	2237	40	133	21320	297521	8″×5″×28
50′-0″	16′-8″	,,	,,	50	208	26650	465296	10″×5″×30
60′-0″	20′-0″	,,	,,	60	300	31980	671100	12″×5″×39
70′-0″	23′-4″	,,	,,	70	408	37310	912696	14″×6″×46
80′-0″	26′-8″	,,	,,	80	533	42640	1192321	15″×6″×59
90′-0″	30′-0″	,,	,,	90	675	47970	1509975	18″×7″×75
100′-0″	33′-4″	,,	,,	100	833	53300	1863421	18″×7″×75
Type 3				**Stancheons 15′-0″ Centres**				
40′-0″	13′-4″	,,	,,	50	167	26650	373579	10″×5″×30
50′-0″	16′-8″	,,	,,	62·5	260	33312	581620	12″×5″×39
60′-0″	20′-0″	,,	,,	75	375	39975	838875	14″×6″×46
70′-0″	23′-4″	,,	,,	87·5	510	46637	1140870	14″×6″×57
80′-0″	26′-8″	,,	,,	100	667	53300	1492079	18″×7″×75
90′-0″	30′-0″	,,	,,	112·5	844	59962	1888028	18″×7″×75
100′-0″	33′-4″	,,	,,	125	1042	66625	2330954	20″×7½″×89

Sheet 4/20

STANDARD SECTIONS FOR SIDE STANCHEONS.

TYPE 4.

SLATED ROOFS.



Type 4				Stancheons 12′-0″ Centres				
Span of Shed	Height of Shed	Unit vert. load	Unit B.M.	Multipliers		Total V.L.	Total B.M.	I Section Required
				V.L.	B.M.			
		lbs.	inch-lbs.			lbs.	inch-lbs.	lbs.
40′-0″	13′-4″	550	1254	40	133	22000	166782	8″×4″×18
50′-0″	16′-8″	„	„	50	208	27500	260832	8″×5″×28
60′-0″	20′-0″	„	„	60	300	33000	376200	10″×5″×30
70′-0″	23′-4″	„	„	70	408	40500	511632	12″×5″×39
80′-0″	26′-8″	„	„	80	533	44000	668382	12″×5″×39
90′-0″	30′-0″	„	„	90	675	49500	846450	14″×6″×46
100′-0″	33′-4″	„	„	100	833	55000	1044582	14″×6″×57
Type 4				Stancheons 15′-0″ Centres				
40′-0″	13′-4″	„	„	50	167	27500	209418	9″×4″×21
50′-0″	16′-8″	„	„	62·5	260	34375	326040	8″×5″×28
60′-0″	20′-0″	„	„	75	375	41250	470250	10″×5″×30
70′-0″	23′-4″	„	„	87·5	510	48125	639540	12″×5″×39
80′-0″	26′-8″	„	„	100	667	55000	836418	14″×6″×46
90′-0″	30′-0″	„	„	112·5	844	61875	1058376	14″×6″×57
100′-0″	33′-4″	„	„	125	1042	68750	1306668	16″×6″×62

Sheet 4/21

STANDARD SECTIONS FOR ROOF PURLINS.

The forces acting on the purlins are those due to wind, to the weight of the purlins, and to the weight of the covering, the amount being found as follows :—

Wind : Normal pressure = 21⅓ lbs. per sup. ft.
Covering : Weight = 12 lbs. per sup. ft.
Purlin : Weight = say, 6½ to 11 lbs. per ft. run according to span.

Assuming a constant spacing of 4′-6″ and also the weight as acting normally to the roof slope (to be correct, the effect of the normal equivalent of the vertical force plus the component at the angle of the roof slope should be taken, but the latter may be considered as taken up by the roof covering, hence the foregoing assumption), then the above forces per foot run will be :—

Wind = 21⅓ × 4′-6″ = 96 lbs.
Covering = 12 × 4′-6″ = 54 lbs.
Purlin = 6½ lbs. to 11 lbs.

CALCULATIONS FOR PURLINS 12′-0″ SPAN.

$$BM = \frac{WL.}{8}$$

$$W = (150 + 6{\cdot}5) \times 12 = 1878 \text{ lbs.}$$

$$BM = \frac{1878 \times 144}{8} = 33{,}804 \text{ inch-lbs.}$$

$$MR = BM = 33{,}804 \text{ inch-lbs.}$$

$$MS = MR \div F.$$

$$F = 16{,}800 \text{ lbs. per sq. inch.}$$

$$MS = \frac{33{,}804}{16{,}800} = 2{\cdot}01.$$

The nearest suitable British Standard Joist Section is 4¾″ × 1¾″ × 6½ lbs. with a maximum MS of 2·85, therefore the actual stress will equal $\frac{33{,}804}{2{\cdot}85} = 11{,}861$ lbs. per sq. inch.

CALCULATIONS FOR PURLINS 15′-0″ SPAN.

$$W = (150 + 11) \times 15 = 2415 \text{ lbs.}$$

$$BM = \frac{2415 \times 180}{8} = 54{,}338 \text{ inch-lbs.}$$

$$MS = \frac{54{,}338}{16{,}800} = 3{\cdot}24.$$

The nearest suitable British Standard Joist Section is 5″ × 3″ × 11 lbs. with a maximum MS of 5·4, therefore the actual stress will equal $\frac{54{,}338}{5{\cdot}4} = 10{,}063$ lbs. per sq. inch.

Owing to the narrowness of the flanges of the joist purlins there will be a tendency to sag in a direction parallel to the rafter, but this objection may be overcome by using a timber fillet of suitable width, i.e. 4" for 12 ft. spans and 5" for 15 ft. spans, such fillet being made of suitable thickness—usually 2"—for the attachment of the roof boards or spars as the case may be.

End Framing.—The method for the determination of the stresses and sections is described in Chapter II, and this method has been adopted in the calculations for the members tabulated on Sheets 4/22 and 4/23 for 40 to 60 ft. spans, and on Sheets 4/24 and 4/25 for 70 to 100 ft. spans.

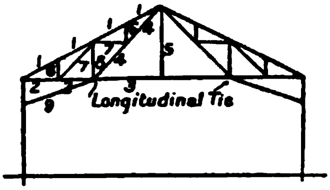
Comparing the arrangements of members for brick-panelled walls, it will be noted that brace "B^3" and strut "S^4" are not considered necessary, the action of these being superseded by the brickwork.

Foundations.—The sizes of the side and gable foundations for the various types and spans are given on Sheet 4/27, and have been calculated for the same earth pressures specified in the previous chapters, i.e. 3 tons per square foot for Types 2 and 4, and 1 ton per square foot for Type 3.

STANDARD SECTIONS FOR SPANS 40 TO 70 FT.

TYPE 2.

SLATED ROOFS.



Member	40′-0″ Span	50′-0″ Span	60′-0″ Span	70′-0″ Span	Centres
1	2/2½″ × 2″ × ¼″ ⅃L (2/3″ × 2½″ × ⁵⁄₁₆″ ⅃L)	2/3″ × 2″ × ¼″ ⅃L (2/3½″ × 2½″ × ⁵⁄₁₆″ ⅃L)	2/3″ × 2½″ × ⁵⁄₁₆″ ⅃L (2/3½″ × 2½″ × ⅜″ ⅃L)	2/3½″ × 2½″ × ⁵⁄₁₆″ ⅃L (2/4″ × 3″ × ⅜″ ⅃L)	12′-0″ Centres.
2	1/3″ × 2″ × ¼″ L	2/2½″ × 2″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	2/3″ × 2″ × ¼″ ⅃L	
3	1/2½″ × 2″ × ¼″ L	2/2½″ × 2″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	
4	1/2½″ × 2″ × ¼″ L	1/2½″ × 2½″ × ¼″ L	1/3″ × 3″ × ¼″ L	2/2½″ × 2″ × ¼″ ⅃L	
5	1/2″ × ¼″ Flat	1/2″ × ¼″ Flat	1/2½″ × ¼″ Flat	1/2½″ × ¼″ Flat	
6	1/2″ × 2″ × ¼″ L	1/2½″ × 2½″ × ¼″ L	1/2½″ × 2½″ × ¼″ L	1/3″ × 3″ × ¼″ L	
7	1/2″ × 2″ × ¼″ L	1/2″ × 2″ × ¼″ L	1/2″ × 2″ × ¼″ L	1/2″ × 2″ × ¼″ L	
8	1/3″ × 3″ × ¼″ L	1/3″ × 3″ × ⁵⁄₁₆″ L	1/3½″ × 3½″ × ⁵⁄₁₆″ L	2/3″ × 2″ × ¼″ ⅃L	
1	2/3″ × 2″ × ¼″ ⅃L (2/3″ × 2½″ × ⁵⁄₁₆″ ⅃L)	2/3″ × 2″ × ⁵⁄₁₆″ ⅃L (2/3½″ × 2½″ × ⅜″ ⅃L)	2/3½″ × 2½″ × ⁵⁄₁₆″ ⅃L (2/4″ × 3″ × ⅜″ ⅃L)	2/4″ × 3″ × ⁵⁄₁₆″ ⅃L (2/5″ × 3″ × ⅜″ ⅃L)	15′-0″ Centres.
2	1/3″ × 2½″ × ¼″ L	2/2½″ × 2″ × ¼″ ⅃L	2/3″ × 2″ × ¼″ ⅃L	2/3″ × 2½″ × ¼″ ⅃L	
3	1/2½″ × 2″ × ¼″ L	2/2½″ × 2″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	
4	1/3″ × 2″ × ¼″ L	1/3″ × 3″ × ¼″ L	2/2½″ × 2″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	
5	1/2″ × ¼″ Flat	1/2″ × ¼″ Flat	1/2½″ × ¼″ Flat	1/2½″ × ¼″ Flat	
6	1/2″ × 2″ × ¼″ L	1/2½″ × 2½″ × ¼″ L	1/2½″ × 2½″ × ¼″ L	1/3″ × 3″ × ¼″ L	
7	1/2″ × 2″ × ¼″ L	1/2″ × 2″ × ¼″ L	1/2″ × 2″ × ¼″ L	1/2″ × 2″ × ¼″ L	
8	1/3″ × 3″ × ¼″ L	1/3″ × 3″ × ⁵⁄₁₆″ L	1/3½″ × 3½″ × ⁵⁄₁₆″ L	2/3″ × 2″ × ¼″ ⅃L	

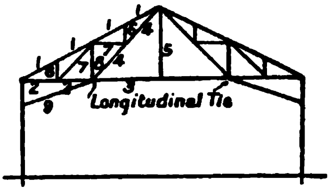
SHEET 4/15

Sections in brackets for Member " 1 " allow for flexural stress due to purlin in addition to " principal " stress.

STANDARD SECTIONS FOR SPANS 40 TO 70 FT.

TYPES 3 AND 4.

SLATED ROOFS.



Member	40′-0″ Span	50′-0″ Span	60′-0″ Span	70′-0″ Span	Centres
1	2/2½″ × 2″ × ¼″ ⅃L (2/3″ × 2½″ × $\frac{5}{16}$″ ⅃L)	2/3″ × 2½″ × ¼″ ⅃L (2/3½″ × 3″ × $\frac{5}{16}$″ ⅃L)	2/3½″ × 2½″ × $\frac{5}{16}$″ ⅃L (2/4″ × 3″ × $\frac{5}{16}$″ ⅃L)	2/3½″ × 3″ × $\frac{5}{16}$″ ⅃L (2/5″ × 3″ × $\frac{5}{16}$″ ⅃L)	12′-0″ Centres.
2	2/2½″ × 2½″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	2/3″ × 2″ × ¼″ ⅃L	
3	2/2½″ × 2½″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	
4	2/2½″ × 2″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	2/3″ × 2″ × ¼″ ⅃L	2/3½″ × 2½″ × ¼″ ⅃L	
5	1/2″ × ¼″ Flat	1/2″ × ¼″ Flat	1/2½″ × ¼″ Flat	1/2½″ × ¼″ Flat	
6	1/2″ × 2″ × ¼″ L	1/2½″ × 2½″ × ¼″ L	1/2½″ × 2½″ × ¼″ L	1/3″ × 3″ × ¼″ L	
7	1/2″ × 2″ × ¼″ L	1/2″ × 2″ × ¼″ L	1/2″ × 2″ × ¼″ L	1/2″ × 2″ × ¼″ L	
8	1/3″ × 3″ × ¼″ L	1/3″ × 3″ × $\frac{5}{16}$″ L	2/2½″ × 2″ × ¼″ ⅃L	2/3″ × 2″ × ¼″ ⅃L	
9	2/3″ × 2″ × ¼″ ⅃L	2/3″ × 2½″ × ¼″ ⅃L	2/3″ × 3″ × ¼″ ┐└	2/3″ × 3″ × $\frac{5}{16}$″ ┐└	
1	2/3″ × 2″ × ¼″ ⅃L (2/3½″ × 2½″ × $\frac{5}{16}$″ ⅃L)	2/3″ × 2½″ × $\frac{5}{16}$″ ⅃L (2/4″ × 3″ × $\frac{5}{16}$″ ⅃L)	2/3½″ × 3″ × $\frac{5}{16}$″ ⅃L (2/4½″ × 3″ × $\frac{5}{16}$″ ⅃L)	2/4″ × 3½″ × $\frac{5}{16}$″ ⅃L (2/5″ × 3″ × ⅜″ ⅃L)	15′-0″ Centres.
2	2/2½″ × 2½″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	2/3″ × 2″ × ¼″ ⅃L	2/3″ × 2½″ × ¼″ ⅃L	
3	2/2½″ × 2½″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	
4	2/2½″ × 2″ × ¼″ ⅃L	2/2½″ × 2″ × ¼″ ⅃L	2/3″ × 2″ × ¼″ ⅃L	2/3½″ × 2½″ × ¼″ ⅃L	
5	1/2″ × ¼″ Flat	1/2″ × ¼″ Flat	1/2½″ × ¼″ Flat	1/2½″ × ¼″ Flat	
6	1/2½″ × 2½″ × ¼″ L	1/3″ × 3″ × ¼″ L	1/3″ × 3″ × ¼″ L	1/3″ × 3″ × $\frac{5}{16}$″ L	
7	1/2″ × 2″ × ¼″ L	1/2″ × 2″ × ¼″ L	1/2″ × 2″ × ¼″ L	1/2″ × 2″ × ¼″ L	
8	1/3″ × 3″ × $\frac{5}{16}$″ L	1/2½″ × 2″ × ¼″ L	2/3″ × 2″ × ¼″ ⅃L	2/3½″ × 2½″ × ¼″ ⅃L	
9	2/3″ × 2″ × ¼″ ⅃L	2/3½″ × 2½″ × ¼″ ⅃L	2/3″ × 3″ × $\frac{5}{16}$″ ┐└	2/3″ × 3″ × ⅜″ ┐└	

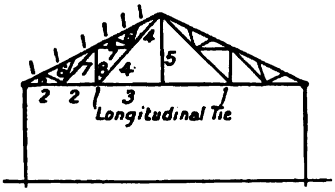
Sheet 4/16

Sections in brackets for Member " 1 " allow for flexural stress due to purlin in addition to "principal" stress.

STANDARD SECTIONS FOR SPANS 70 TO 100 FT.

TYPE 2.

SLATED ROOFS.



Member	70′-0″ Span	80′-0″ Span	90′-0″ Span	100′-0″ Span	Centres
1	2/3½″×2½″×$\frac{5}{16}$″ ⅃L (2/4″ ×3″ ×$\frac{5}{16}$″ ⅃L)	2/3½″×3″ ×$\frac{5}{16}$″ ⅃L (2/5″ ×3″ ×$\frac{5}{16}$″ ⅃L)	2/4″ ×3½″×$\frac{5}{16}$″ ⅃L (2/5″ ×3″ ×⅜″ ⅃L)	2/5″ ×3″ ×$\frac{5}{16}$″ ⅃L (2/6″ ×3″ ×⅜″ ⅃L)	12′-0″ Centres.
2	2/2½″×2″ ×¼″ ⅃L	2/3″ ×2″ ×¼″ ⅃L	2/3″ ×2½″×¼″ ⅃L	2/3½″×2½″×¼″ ⅃L	
3	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	
4	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	2/3″ ×2″ ×¼″ ⅃L	2/3″ ×2″ ×¼″ ⅃L	
5	1/2½″×¼″ Flat	1/2½″×¼″ Flat	1/2½″×$\frac{5}{16}$″ Flat	1/2½″×$\frac{5}{16}$″ Flat	
6	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	2/3″ ×2″ ×¼″ ⅃L	2/3½″×2½″×¼″ ⅃L	
7	1/2″ ×2″ ×¼″ L	1/2½″×2″ ×¼″ L	1/2½″×2″ ×¼″ L	1/3″ ×2″ ×¼″ L	
8	2/3″ ×2″ ×¼″ ⅃L	2/3″ ×2½″×¼″ ⅃L	2/3½″×2½″×¼″ ⅃L	2/3½″×3″ ×$\frac{5}{16}$″ ⅃L	
1	2/3½″×2½″×$\frac{5}{16}$″ ⅃L (2/4″ ×3½″×$\frac{5}{16}$″ ⅃L)	2/3½″×3″ ×⅜″ ⅃L (2/5″ ×3″ ×⅜″ ⅃L)	2/4″ ×3½″×⅜″ ⅃L (2/5″ ×3″ ×$\frac{7}{16}$″ ⅃L)	2/5″ ×3″ ×⅜″ ⅃L (2/6″ ×3″ ×½″ ⅃L)	15′-0″ Centres.
2	2/3″ ×2½″×¼″ ⅃L	2/3″ ×2″ ×$\frac{5}{16}$″ ⅃L	2/3″ ×2½″×$\frac{5}{16}$″ ⅃L	2/3½″×2½″×$\frac{5}{16}$″ ⅃L	
3	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	
4	2/3″ ×2″ ×¼″ ⅃L	2/3″ ×2½″×¼″ ⅃L	2/3½″×2½″×¼″ ⅃L	2/3″ ×2½″×$\frac{5}{16}$″ ⅃L	
5	1/2½″×¼″ Flat	1/2½″×¼″ Flat	1/2½″×$\frac{5}{16}$″ Flat	1/2½″×$\frac{5}{16}$″ Flat	
6	2/2½″×2″ ×¼″ ⅃L	2/3″ ×2″ ×¼″ ⅃L	2/3″ ×2½″×¼″ ⅃L	2/3½″×2½″×¼″ ⅃L	
7	1/2½″×2″ ×¼″ L	1/3″ ×2″ ×¼″ L	1/3″ ×2½″×¼″ L	1/3½″×2½″×¼″ L	
8	2/3″ ×2″ ×$\frac{5}{16}$″ ⅃L	2/3″ ×2½″×$\frac{5}{16}$″ ⅃L	2/3½″×2½″×$\frac{5}{16}$″ ⅃L	2/3½″×3″ ×$\frac{5}{16}$″ ⅃L	

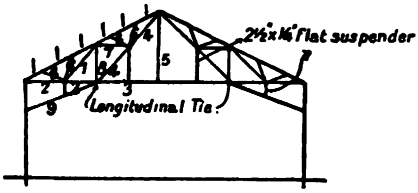
Sheet 4/17

Sections in brackets for Member " 1 " allow for flexural stress due to purlin in addition to " principal " stress.

STANDARD SECTIONS FOR SPANS 70 TO 100 FT.

TYPES 3 AND 4.

SLATED ROOFS.



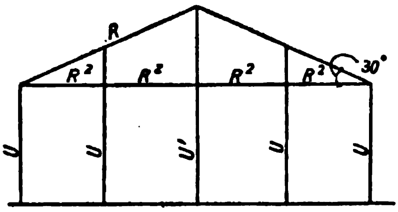
Member	70′-0″ Span	80′-0″ Span	90′-0″ Span	100′-0″ Span	Centres
1	2/3½″×2½″×5/16″ ⅃L (2/4″ ×3″ ×5/16″ ⅃L)	2/3½″×3″ ×5/16″ ⅃L (2/5″ ×3″ ×5/16″ ⅃L)	2/5″×3″ ×5/16″ ⅃L (2/5″ ×3½″×⅜″ ⅃L)	2/5″ ×4″ ×5/16″ ⅃L (2/6″ ×3½″×⅜″ ⅃L)	12′-0″ Centres.
2	2/3″ ×2″ ×¼″ ⅃L	2/3″ ×2½″×¼″ ⅃L	2/3½″×2½″×¼″ ⅃L	2/3″ ×2″ ×5/16″ ⅃L	
3	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	
4	2/3½″×2½″×¼″ ⅃L	2/4″ ×2½″×¼″ ⅃L	2/4″ ×3″ ×5/16″ ⅃L	2/4½″×3½″×5/16″ ⅃L	
5	1/2½″×¼″ Flat	1/2½″×¼″ Flat	1/2½″×5/16″ Flat	1/2½″×5/16″ Flat	
6	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	2/3″ ×2″ ×¼″ ⅃L	2/3½″×2½″×¼″ ⅃L	
7	1/2″ ×2″ ×¼″ L	1/2½″×2″ ×¼″ L	1/2½″×2″ ×¼″ L	1/3″ ×2″ ×¼″ L	
8	2/3″ ×2″ ×¼″ ⅃L	2/3″ ×2½″×¼″ ⅃L	2/3½″×2½″×¼″ ⅃L	2/3½″×3″ ×5/16″ ⅃L	
9	2/3″ ×3″ ×¼″ ┘┌	2/3½″×3½″×5/16″ ┘┌	2/4″ ×4″ ×5/16″ ┘┌	2/5″ ×4″ ×5/16″ ┘┌	
1	2/3½″×2½″×⅜″ ⅃L (2/4″ ×3½″×⅜″ ⅃L)	2/3½″×3″ ×⅜″ ⅃L (2/6″ ×3″ ×⅜″ ⅃L)	2/5″ ×3″ ×⅜″ ⅃L (2/6″ ×3″ ×7/16″ ⅃L)	2/5″ ×4″ ×⅜″ ⅃L (2/6″ ×3½″×½″ ⅃L)	15′-0″ Centres.
2	2/3″ ×2″ ×5/16″ ⅃L	2/3″ ×2½″×5/16″ ⅃L	2/3½″×2½″×5/16″ ⅃L	2/3½″×3″ ×5/16″ ⅃L	
3	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	2/2½″×2″ ×¼″ ⅃L	
4	2/3½″×2½″×¼″ ⅃L	2/4″ ×2½″×¼″ ⅃L	2/4″ ×3″ ×5/16″ ⅃L	2/4½″×3½″×5/16″ ⅃L	
5	1/2½″×5/16″ Flat	1/2½″×5/16″ Flat	1/2½″×5/16″ Flat	1/2½″×5/16″ Flat	
6	2/2½″×2″ ×¼″ ⅃L	2/3″ ×2″ ×¼″ ⅃L	2/3″ ×2½″×¼″ ⅃L	2/3½″×2½″×¼″ ⅃L	
7	1/2½″×2″ ×¼″ L	1/3″ ×2″ ×¼″ L	1/3″ ×2½″×¼″ L	1/3½″×2½″×¼″ L	
8	2/3″ ×2″ ×5/16″ ⅃L	2/3″ ×2½″×5/16″ ⅃L	2/3½″×2½″×5/16″ ⅃L	2/3½″×3″ ×5/16″ ⅃L	
9	2/3″ ×3″ ×5/16″ ┘┌	2/3½″×3½″×⅜″ ┘┌	2/4″ ×4″ ×⅜″ ┘┌	2/5″ ×4″ ×⅜″ ┘┌	

SHEET 4/18

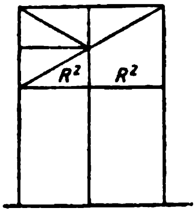
Sections in brackets for Member " 1 " allow for flexural stress due to purlin in addition to "principal" stress.

SECTIONS FOR 40 FT. TO 60 FT. SPAN GABLE FRAMES AND BRACINGS TO SUIT PRINCIPALS 12 FT. CENTRES.

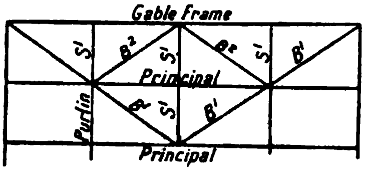
SLATED ROOFS.



Front Elevation.



Side Elevation.



Plan.

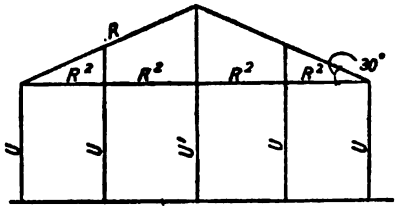
MEMBER	40′-0″ SPAN	50′-0″ SPAN	60′-0″ SPAN
Upright U	9″ × 4″ × 21 lbs. I	10″ × 5″ × 30 lbs. I	12″ × 5″ × 32 lbs. I
,, U^1	10″ × 5″ × 30 lbs. I	12″ × 5″ × 32 lbs. I	12″ × 6″ × 44 lbs. I
Rafter R	1/4″ × 3″ × ⅜″ L	1/5″ × 3″ × ⅜″ L	1/6″ × 4″ × ⅜″ L
Strut S^1	2/3″ × 2″ × ¼″ ┐┌	2/3″ × 2″ × ¼″ ┐┌	2/3″ × 2½″ × ¼″ ┐┌
Rail R^2	1/6″ × 2½″ ⊓	1/6″ × 2½″ ⊓	1/6″ × 2½″ ⊓
Brace B^1	1/2½″ × 2″ × ¼″ L	1/3″ × 2″ × ¼″ L	1/3″ × 3″ × ¼″ L
,, B^2	1/2½″ × 2″ × ¼″ L	1/3″ × 2″ × ¼″ L	1/3″ × 2″ × ¼″ L

SHEET 4/22

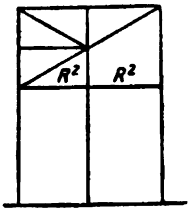
NOTE.—Uprights U and U^1 will suit a 9″ wall best if the depth is not less than 12″.

SECTIONS FOR 40 FT. TO 60 FT. SPAN GABLE FRAMES AND BRACINGS TO SUIT PRINCIPALS 15 FT. CENTRES.

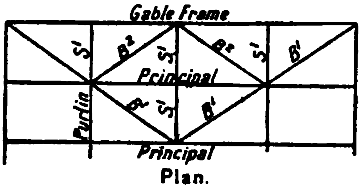
SLATED ROOFS.



Front Elevation.



Side Elevation.



Plan.

MEMBER	40′-0″ SPAN	50′-0″ SPAN	60′-0″ SPAN
Upright U	9″ × 4″ × 21 lbs. ⌶	10″ × 5″ × 30 lbs. ⌶	12″ × 5″ × 32 lbs. ⌶
„ U^1	10″ × 5″ × 30 lbs. ⌶	12″ × 5″ × 32 lbs. ⌶	12″ × 6″ × 44 lbs. ⌶
Rafter R	1/4″ × 3″ × ⅜″ ∟	1/5″ × 3″ × ⅜″ ∟	1/6″ × 4″ × ⅜″ ∟
Strut S^1	2/3½″ × 2½″ × ¼″ ┐┌	2/3½″ × 2½″ × ¼″ ┐┌	2/3″ × 2½″ × ¼″ ┐┌
Rail R^2	1/6″ × 3″ ⊓	1/6″ × 3″ ⊓	1/6″ × 3″ ⊓
Brace B^1	1/2½″ × 2″ × ¼″ ∟	1/3″ × 2″ × ¼″ ∟	1/3″ × 3″ × ¼″ ∟
„ B^2	1/2½″ × 2″ × ¼″ ∟	1/3″ × 2″ × ¼″ ∟	1/3″ × 2″ × ¼″ ∟

SHEET 4/23

NOTE.—Uprights U and U^1 will suit a 9″ wall best if the depth is not less than 12″.

CHAPTER V.

STANDARD SECTIONS FOR SHEDS WITH OPEN SIDES AND GABLES. SLATED ROOFS.

THE procedure of this chapter follows very closely that of Chapter III, whilst both chapters are an adaptation of the one preceding each.

Adopting the same symbols and data as last chapter, the force constants for, and amount of, reactions will be as follows :—

Type 2, Principals (30° pitch).

Total upward reaction at $\mathrm{WRP} = \frac{W}{2} + {\cdot}571\ \mathrm{NP} = 125{\cdot}82 + 84{\cdot}8$ $= 210{\cdot}08$ lbs.

Total upward reaction at $\mathrm{LRP} = \frac{W}{2} + {\cdot}289\ \mathrm{NP} = 125{\cdot}82 + 42{\cdot}9$ $= 168{\cdot}18$ lbs.

Total horizontal reaction at WRP and $\mathrm{LRP} = {\cdot}25\ \mathrm{NP} = {\cdot}25 \times 148{\cdot}5$ $= 37{\cdot}125$ lbs.

Type 3, Transverse Frames.

Total upward reaction at $\mathrm{WRT} = \frac{W}{2} + \left(\frac{C - ({\cdot}3364C + {\cdot}577S^2)}{C} \times \mathrm{VN}\right)$ $= 125{\cdot}28 + 60{\cdot}3 = 185{\cdot}58$ lbs.

Total upward reaction at $\mathrm{LRT} = \frac{W}{2} + \left(\frac{{\cdot}3364C + {\cdot}577S^2}{C} \times \mathrm{VN}\right) = 125{\cdot}28$ $+ 67{\cdot}3 = 192{\cdot}58$ lbs.

Total horizontal reaction at WRT and $\mathrm{LRT} = {\cdot}25\ \mathrm{NP} = 37{\cdot}125$ lbs.

Type 4, Transverse Frames.

Total upward reaction at $\mathrm{WRT} = \frac{W}{2} + \left(\frac{C - ({\cdot}3364C + {\cdot}577S^2)}{C} \times \mathrm{VN}\right)$ $= 125{\cdot}28 + 69{\cdot}39 = 194{\cdot}67$ lbs.

Total upward reaction at $\mathrm{LRT} = \frac{W}{2} + \left(\frac{{\cdot}3364C + 577S^2}{C} \times \mathrm{VN}\right) = 125{\cdot}28$ $+ 58{\cdot}308 = 183{\cdot}588$ lbs.

Total horizontal reaction at WRT and $\mathrm{LRT} = {\cdot}25\ \mathrm{NP} = 37{\cdot}125$ lbs.

The only difference in the stresses in the principal members for the two classes of sheds, i.e. open and closed sheds, is that in the bottom chord, such difference being due to the absence (in the open-shed type) of that portion of the side-wind force—acting on the closed-shed type—transmitted from the windward to the leeward side. The effect of this force being that of counteracting and therefore reducing the tensile force in the bottom chord, it follows that the total stress in the latter member may be found by adding this force to the tabulated stresses given in the preceding chapter, and as the tabulated sections have, for practical reasons, been made sufficient for this purpose, the sections given for the closed type may be adopted for use in the open type of shed.

Side Stancheons.—The horizontal forces, and therefore the bending moments, are of course much less for the open type than for the closed type sheds. The unit stresses for the various types are given hereafter.

Type 2.—The maximum bending moment occurs at the ground-level, the unit amount being found as follows :—

Maximum vertical load occurs at WRP and equals 210·08 lbs.
Horizontal reaction at WRP = 37·125 lbs.
Leverage of do. = 4′-0″.
BM = 37·125 × 48″ = 1782 inch-lbs.

Type 3.—The maximum bending moment occurs at the connection of the stay to the stancheon, the unit amount being found as follows :—

Maximum vertical load occurs at LRT and equals 192·58 lbs.
Horizontal reaction at LRT = 37·125 lbs.
Leverage of do. = 3′-0″.
BM = 37·125 × 36″ = 1336·5 inch-lbs.

Type 4.—The maximum bending moment occurs at the connection of the stay to the stancheon as for Type 3, the unit amount being found as follows :—

Maximum vertical load occurs at WRT and equals 194·67 lbs.
Horizontal reaction at WRT = 37·125 lbs.
Leverage of do. = 1′-6″.
BM = 37·125 × 18″ = 668·25 inch-lbs.

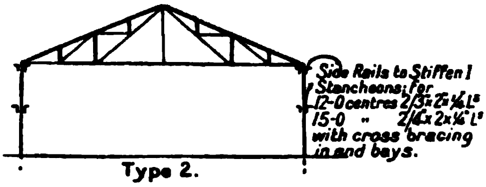
Roof Bracing.—The functions of the roof bracing are the prevention of side flexure in the principal rafters and the stiffening of the framework during construction. For these purposes it is only necessary to use braces B^1 and B^2, and the sections of these members may be similar to those adopted for the closed type of sheds.

Side Braces.—The function of these members is to eliminate distortion in the transverse frames due to the action of the horizontal forces acting at the reaction points WRT and LRT.

STANDARD SECTIONS FOR SIDE STANCHEONS.

TYPE 2.

OPEN SHEDS. SLATED ROOFS.


Side Rails to Stiffen I Stancheons; for
12-0 centres 2/3″×2″×¼″ L^s
15-0 " 2/4″×2×¼″ L^s
with cross bracing
in end bays.

Type 2.

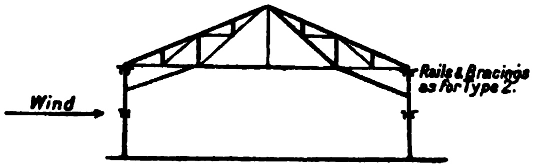
Span of Shed	Height of Shed	Unit vert. load	Unit B.M.	Multipliers		Total V.L.	Total B.M.	I Section Required
				V.L.	B.M.			
Type 2				**Stancheons 12-0″ Centres**				
		lbs.	inch-lbs.			lbs.	inch-lbs.	lbs.
40′-0″	13′-4″	210	1782	40	133	8400	237006	8″×5″×28
50′-0″	16′-8″	,,	,,	50	208	10500	370656	12″×5″×32
60′-0″	20′-0″	,,	,,	60	300	12600	534600	12″×6″×44
70′-0″	23′-4″	,,	,,	70	408	14700	727056	14″×6″×46
80′-0″	26′-8″	,,	,,	80	533	16800	949806	16″×6″×62
90′-0″	30′-0″	,,	,,	90	675	18900	1202850	18″×7″×75
100′-0″	33′-4″	,,	,,	100	833	21000	1484406	18″×7″×75
Type 2				**Stancheons 15′-0″ Centres**				
40′-0″	13′-4″	,,	,,	50	167	10500	297594	10″×5″×30
50′-0″	16′-8″	,,	,,	62·5	260	13125	463320	12″×5″×39
60′-0″	20′-0″	,,	,,	75	375	15750	668250	14″×6″×46
70′-0″	23′-4″	,,	,,	87·5	510	18375	908820	14″×6″×57
80′-0″	26′-8″	,,	,,	100	667	21000	1188594	18″×7″×75
90′-0″	30′-0″	,,	,,	112·5	844	23625	1504008	18″×7″×75
100′-0″	33′-4″	,,	,,	125	1042	26250	1856844	20″×7½″×89

Sheet 5/1

STANDARD SECTIONS FOR SIDE STANCHEONS.

TYPE 3.

OPEN SHEDS. SLATED ROOFS.



Type 3.

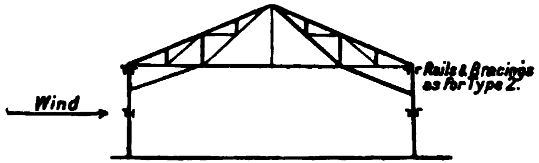
Span of Shed	Height of Shed	Unit vert. load	Unit B.M.	Multipliers V.L.	Multipliers B.M.	Total V.L.	Total B.M.	I Section Required
Type 3				**Stancheons 12′-0″ Centres**				
		lbs.	inch-lbs.			lbs.	inch-lbs.	lbs.
40′-0″	13′-4″	193	1337	40	133	7720	177821	9″×4″×21
50′-0″	16′-8″	„	„	50	208	9650	278096	10″×5″×30
60′-0″	20′-0″	„	„	60	300	11580	401100	12″×5″×32
70′-0″	23′-4″	„	„	70	408	13510	545496	12″×6″×44
80′-0″	26′-8″	„	„	80	533	15440	712621	14″×6″×46
90′-0″	30′-0″	„	„	90	675	17370	902275	14″×6″×57
100′-0″	33′-4″	„	„	100	833	19300	1113721	16″×6″×62
Type 3				**Stancheons 15′-0″ Centres**				
40′-0″	13′-4″	„	„	50	167	9650	223279	8″×5″×28
50′-0″	16′-8″	„	„	62·5	260	12063	347620	12″×5″×32
60′-0″	20′-0″	„	„	75	375	14475	501375	12″×6″×44
70′-0″	23′-4″	„	„	87·5	510	16888	681870	14″×6″×46
80′-0″	26′-8″	„	„	100	667	19300	891779	14″×6″×57
90′-0″	30′-0″	„	„	112·5	844	21713	1128425	16″×6″×62
100′-0″	33′-4″	„	„	125	1042	24125	1393154	18″×7″×75

Sheet 5/2

STANDARD SECTIONS FOR SIDE STANCHEONS.

TYPE 4.

OPEN SHEDS. SLATED ROOFS.



Type 3.

Type 4				Stancheons 12′-0″ Centres				
Span of Shed	Height of Shed	Unit vert. load	Unit B.M.	Multipliers		Total V.L.	Total B.M.	I Section Required
				V.L.	B.M.			
		lbs.	inch-lbs.			lbs.	inch-lbs.	lbs.
40′-0″	13′-4″	195	669	40	133	7800	88977	7″×4″×16
50′-0″	16′-8″	„	„	50	208	9750	139152	8″×4″×18
60′-0″	20′-0″	„	„	60	300	11700	200700	8″×5″×28
70′-0″	23′-4″	„	„	70	408	13650	272952	10″×5″×30
80′-0″	26′-8″	„	„	80	533	15600	356577	12″×5″×32
90′-0″	30′-0″	„	„	90	675	17550	451575	12″×5″×39
100′-0″	33′-4″	„	„	100	833	19500	557277	12″×6″×44
Type 4				Stancheons 15′-0″ Centres				
40′-0″	13′-4″	„	„	50	167	9750	111723	8″×4″×18
50′-0″	16′-8″	„	„	62·5	260	12188	173940	9″×4″×21
60′-0″	20′-0″	„	„	75	375	14625	250875	10″×5″×30
70′-0″	23′-4″	„	„	87·5	510	17063	341190	12″×5″×32
80′-0″	26′-8″	„	„	100	667	19500	446223	12″×5″×39
90′-0″	30′-0″	„	„	112·5	844	21938	564636	12″×6″×44
100′-0″	33′-4″	„	„	125	1042	24375	697098	14″×6″×46

Sheet 5/3

If S^1 = distance from WRP to connection of stay to stancheon,

S^2 = distance of WRP from WRT.

RHP = horizontal force acting at WRT.

A = angle of stay to horizontal = $18\frac{1}{2}°$.

Then the stress in the stay will be tensile at the windward side and compressive at the leeward side, and the amount will be :—

$$\frac{\text{RHP} \times S^2}{S^1} \times \text{Sec. A.}$$

The unit stress in the stay for Types 3 and 4 is as follows :—

Type 3.—

$$\frac{\text{RHP} \times S^2}{S^1} \times \text{Sec. A} = \frac{37{\cdot}125 \times 4}{1} \times 1{\cdot}054 = 156{\cdot}52 \text{ lbs.}$$

Type 4.—

$$\frac{\text{RHP} \times S^2}{S^1} \times \text{Sec. A} = \frac{37{\cdot}125 \times 2\frac{1}{2}}{1} \times 1{\cdot}054 = 97{\cdot}825 \text{ lbs.}$$

The total stress for the various spans may be obtained by multiplying the above unit stresses by the multipliers given in the preceding chapter for the roof members.

Although the unit stresses given above are much less than those for the closed type of shed, the sections given for the latter will apply here, as these sections were determined by the ratio of $\frac{L}{R}$, which, of course, remains the same.

Roof Purlins.—The sections of the purlins will be exactly like those for the closed type of shed.

Side Foundations.—The sizes of the foundations for the various types and spans are given on Sheet 5/4, and have been calculated for the same earth pressures previously adopted.

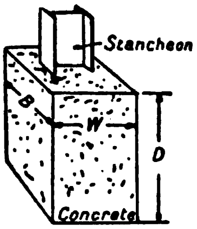
CONCRETE FOUNDATIONS.

OPEN SHEDS. SLATED ROOFS.

Basis : Earth pressure for Types 2 and 4 : 3 tons per sq. ft.
„ „ „ Type 3 : 1 ton „ „

For Side Stancheons 12′-0″ Centres

Span	Height	Type 2	Type 3	Type 4
		D W B	B W D	D W B
40′-0″	13′-4″	3′-6″ × 2′-0″ × 2′-0″	2′-0″ × 2′-0″ × 1′-3″	2′-6″ × 2′-0″ × 2′-0″
50′-0″	16′-8″	4′-0″ × 2′-2″ × 2′-2″	2′-2″ × 2′-2″ × 1′-4″	2′-9″ × 2′-2″ × 2′-2″
60′-0″	20′-0″	4′-6″ × 2′-4″ × 2′-4″	2′-4″ × 2′-4″ × 1′-6″	3′-0″ × 2′-3″ × 2′-3″
70′-0″	23′-4″	5′-0″ × 2′-6″ × 2′-6″	2′-6″ × 2′-6″ × 1′-6″	3′-3″ × 2′-4″ × 2′-4″
80′-0″	26′-8″	5′-6″ × 2′-8″ × 2′-8″	2′-8″ × 2′-8″ × 1′-8″	3′-6″ × 2′-6″ × 2′-6″
90′-0″	30′-0″	6′-0″ × 2′-10″ × 2′-10″	2′-10″ × 2′-10″ × 1′-10″	3′-9″ × 2′-8″ × 2′-8″
100′-0″	33′-4″	6′-6″ × 3′-0″ × 3′-0″	3′-0″ × 3′-0″ × 2′-0″	4′-0″ × 2′-9″ × 2′-9″
For Side Stancheons 15′-0″ Centres				
40′-0″	13′-4″	3′-6″ × 2′-4″ × 2′-4″	2′-6″ × 2′-6″ × 1′-3″	2′-6″ × 2′-6″ × 2′-6″
50′-0″	16′-8″	4′-0″ × 2′-6″ × 2′-6″	2′-9″ × 2′-9″ × 1′-4″	2′-9″ × 2′-8″ × 2′-8″
60′-0″	20′-0″	4′-6″ × 2′-9″ × 2′-9″	3′-0″ × 3′-0″ × 1′-6″	3′-0″ × 2′-10″ × 2′-10″
70′-0″	23′-4″	5′-0″ × 3′-0″ × 3′-0″	3′-2″ × 3′-2″ × 1′-6″	3′-3″ × 3′-0″ × 3′-0″
80′-0″	26′-8″	5′-6″ × 3′-2″ × 3′-2″	3′-4″ × 3′-4″ × 1′-8″	3′-6″ × 3′-3″ × 3′-3″
90′-0″	30′-0″	6′-0″ × 3′-4″ × 3′-4″	3′-6″ × 3′-6″ × 1′-10″	3′-9″ × 3′-4″ × 3′-4″
100′-0″	33′-4″	6′-6″ × 3′-6″ × 3′-6″	3′-9″ × 3′-9″ × 2′-0″	4′-0″ × 3′-6″ × 3′-6″



CHAPTER VI.

STANDARD STRESSES AND SECTIONS FOR RIDGED-ROOF PRINCIPALS.

Types of Roof Principals.—Primarily, a ridged-roof principal is composed of two inclined rafters reacting against each other at their apex; prevented from spreading at the supports by means of tie-rods connected to each shoe; and supported by means of walls, girders, stancheons, or other means.

With the principal simply loaded at the apex, and at each shoe, as is often the case with principals of small span, the effect of the load is to mutually apply pressure at the support, and also set up stresses in the rafters and tie. The nature of the stresses thus created is compressive in the rafter and tensile in the tie. These stresses may be termed "Principal Stresses."

As the span increases, it becomes uneconomical to distribute the wind and roof loads direct to the apex and shoes, consequently these forces are applied at one or more intermediate points as well as at the extremities of the rafters. This distribution of forces and consequent transmission of these forces to the extremities of the rafters sets up bending action in the rafter, and the stress thus created in the rafter supplements the "Principal Stress" referred to above.

This distribution of forces is most economically resisted by bracing the rafter, the arrangement of bracing being to a large extent dependent upon the number of points at which these forces are applied; hence for spans of over 15 feet the simple truss is transformed into two inclined triangular-braced girders reacting against each other at their apex and prevented from spreading, and consequently thrusting out the walls or supports at their shoes. There are many types of ridged principals in use, all of which, however (that is, all principals having the main tie horizontal or nearly horizontal), have evolved along these lines. This chapter will deal with the standardization of stresses and sections for types in common use, the outlines of which are given later.

Reverting to the distribution of the roof forces over the rafter, the point at which the force is applied should, where possible, coincide with the intersection of the lines of the centre of action of the braces with that of the rafter, that is, over a strut, otherwise flexural stress will be set up in the rafter.

Stresses.—Recapitulating, and ignoring secondary stresses set up by changes of temperature, and the bending action due to the weight of the members spanning from joint to joint, the stresses set up in, and to be resisted by, the principal members are :—

(1) The "principal" or direct stresses in the rafters, and tie from shoe to shoe, due to the transmission to the supports of the apex load.

(2) The secondary stresses set up in the braced rafters in transmitting the intermediate loads to the extremities of the rafters.

(3) The further secondary stresses set up in the rafter, due to the bending action created by the situation of the purlin, if not placed directly over a strut.

Stresses (1) and (2) are proportional to the span and distance apart of the principals, whilst stress (3) is variable according to the amount of the load from the purlin, and its position relative to that of the struts. Unit stresses may be calculated for stresses (1) and (2), from which the total stresses may easily be obtained. The lengths of the various members also vary as the span; therefore, unit lengths may be calculated, from which the total length to suit any particular span may be easily found.

The unit stresses and unit lengths for the various members of principals of different types and spans are given on Sheets 6/1 to 6/4 for roofs sheeted or slated, and the total stresses on Sheets 6/5 to 6/8. These stresses have been obtained from stress diagrams prepared in accordance with the following data :—

(1) Slope of roof for sheeted type taken as 24½°, and for slated type as 30°.

(2) Span taken as the distance between the intersections—at either shoe—of the lines of the centre of action of the rafter, main tie, and support.

(3) Wind pressure taken as 32 lbs. per square foot of roof surface of one slope, and acting in a horizontal direction, but reduced to its normal equivalent pressure per square foot, i.e. 18 lbs. for sheeted roofs (24½° slope), and 21⅓ lbs. for slated roofs (30° slope), the wind pressure in each case being equally distributed over the whole of one slope of the roof.

(4) Dead load per square foot for sheeted roofs made up as follows :—

Roof principal . .	3½ lbs.	Total 8 lbs.
Purlins . . .	1½ ,,	
Roof covering . .	3 ,,	

Dead load per square foot for slated roofs made up as follows :—

Roof principal . .	4 lbs.	Total 18 lbs.
Purlins . . .	2 ,,	
Roof covering . .	12 ,,	

These loads are equally distributed over both sides of the roof.

(5) The effect of snow—owing to the high-wind pressure taken which is considered sufficient to clear the roof of snow—is ignored.

(6) Purlins to be placed directly over a strut.

(7) Shoes of principals considered as fixed; therefore the amount of wind and dead load, transmitted to each support, is inversely proportional to the distance from the supports of the point of intersection, of the resultant wind or dead load, with a horizontal line drawn through the reaction points. This method of finding the reaction is the most correct assumption, and is the one in general use.

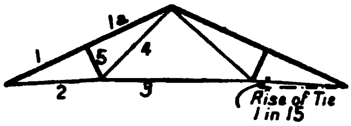
(8) Stresses due to changes of temperature, or initial stresses of any kind, are ignored.

Sections.—The sections given on Sheets 6/9 to 6/12 have been determined by using the safe stresses given hereafter. Safe tensile stress taken as 16,800 lbs. (7½ tons) per square inch of net section.

Safe compressive stress taken at 14,560 lbs. (6½ tons) per square inch of gross section, reduced by strut formula according to the ratio of $\frac{L}{R}$, which is considered as limited to $\frac{160.}{1}$

UNIT STRESSES FOR ORDINARY PRINCIPALS.

SPANS 20 TO 25 FT.



Slope of roof :—
Slated roofs 30° slope.
Sheeted „ 24½° „

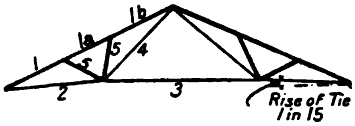
Member	Condition of Ends	Slated Type		Sheeted Type	
		Unit Length	Unit stress in pounds	Unit Length	Unit stress in pounds
1	B.E.F.	3·464	+438	3·2968	+250
1a	B.E.F.	3·464	+413	3·2968	+239
2		3·8618	−428	3·524	−250
3		4·2934	−225	4·968	−126
4		3·8618	−210	3·524	−127
5	O.E.F.	1·7032	+130	1·245	+ 83

Sheet 6/1

+ denotes compression.
— denotes tension.
B.E.F. denotes both ends fixed.
O.E.F. denotes one end fixed, other end free.
Unit length $\times \frac{\text{span}}{12}$ = length of member.

UNIT STRESSES FOR ORDINARY PRINCIPALS.

SPANS 25 TO 40 FT.



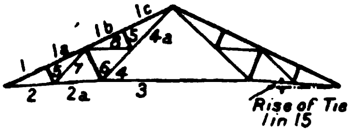
Slope of roof :—
Slated roofs 30° slope.
Sheeted „ 24½° „

Member	Condition of Ends	Slated Type		Sheeted Type	
		Unit Length	Unit stress in pounds	Unit Length	Unit stress in pounds
1	B.E.F.	2·309	+340	2·1979	+298
1a	B.E.F.	2·309	+267	2·1979	+240
1b	B.E.F.	2·309	+296	2·1979	+282
2		3·8618	−332	3·524	−300
3		4·2934	−155	4·968	−128
4		3·8618	−170	3·524	−174
5	O.E.F.	2·058	+100	1·666	+ 77

Sheet 6/2

+ denotes compression.
— denotes tension.
B.E.F. denotes both ends fixed.
O.E.F. denotes one end fixed, other end free.
Unit length $\times \frac{\text{span}}{12}$ = length of member.

UNIT STRESSES FOR ORDINARY PRINCIPALS.
SPANS 40 TO 60 FT.



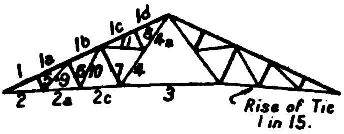
Slope of roof :—
Slated roofs 30° slope.
Sheeted „ 24½° „

Member	Condition of Ends	Slated Type		Sheeted Type	
		Unit Length	Unit stress in pounds	Unit Length	Unit stress in pounds
1	B.E.F.	1·732	+420	1·6484	+316
1a	B.E.F.	1·732	+405	1·6484	+312
1b	B.E.F.	1·732	+390	1·6484	+306
1c	B.E.F.	1·732	+373	1·6484	+300
2		1·9309	—405	1·762	—316
2a		1·9309	—332	1·762	—256
3		4·2934	—175	4·968	—128
4		1·9309	—165	1·762	—132
4a		1·9309	—240	1·762	—192
5	O.E.F.	·8516	+ 65	·6225	+ 41
6	O.E.F.	1·7032	+130	1·245	+ 82
7		1·9309	— 75	1·762	— 60
8		1·9309	— 75	1·762	— 60

+ denotes compression.
— denotes tension.
B.E.F. denotes both ends fixed.
O.E.F. denotes one end fixed, other end free.
Unit length × $\frac{\text{span}}{12}$ = length of member.

Sheet 6/3

UNIT STRESSES FOR ORDINARY PRINCIPALS. SPANS 60 TO 80 FT.



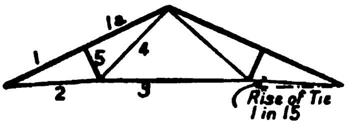
Slope of roof :—Slated roofs 30° slope.
Sheeted „ 24½° „

Member	Condition of Ends	Slated Type		Sheeted Type	
		Unit Length	Unit stress in pounds	Unit Length	Unit stress in pounds
1	B.E.F.	1·385	+425	1·3187	+351
1a	B.E.F.	1·385	+413	1·3187	+347
1b	B.E.F.	1·385	+346	1·3187	+298
1c	B.E.F.	1·385	+318	1·3187	+278
1d	B.E.F.	1·385	+305	1·3187	+274
2		1·5457	−410	1·4096	−351
2a		1·5457	−352	1·4096	−303
2c		1·5457	−294	1·4096	−255
3		2·746	−170	3·562	−143
4		1·722	−145	1·516	−127
4a		1·722	−190	1·516	−161
5	O.E.F.	·6613	+ 53	·4978	+ 33
6	O.E.F.	1·3626	+ 80	·9956	+ 50
7	O.E.F.	2·0429	+130	1·4934	+ 84
8	O.E.F.	1·0219	+ 53	·7467	+ 33
9		1·5457	− 60	1·4096	− 48
10		1·888	− 75	1·653	− 60
11		1·722	− 45	1·516	− 35

+ denotes compression.
— denotes tension.
B.E.F. denotes both ends fixed.
O.E.F. denotes one end fixed, other end free.
Unit length $\times \frac{\text{span}}{12}$ = length of member.

Sheet 6/4

TOTAL STRESSES FOR ORDINARY PRINCIPALS.
SPANS 20 TO 25 FT.



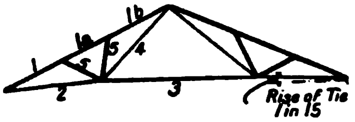
Slope of roof :—
Slated roofs 30° slope.
Sheeted „ 24½° „

Member	Slated Type				Sheeted Type			
	20 ft. Span		25 ft. Span		20 ft. Span		25 ft. Span	
	12′-0″ centres	15′-0″ centres	12′-0″ centres	15′-0″ centres	12′-0″ centres	15′-0″ centres	12′-0″ centres	15′-0″ centres
1	+8760	+10950	+10950	+13685	+5000	+6265	+6250	+7813
1a	+8260	+10325	+10325	+12906	+4780	+5975	+5975	+7469
2	−8560	−10700	−10700	−13375	−5000	−6265	−6250	−7813
3	−4500	− 5625	− 5625	− 7032	−2520	−3150	−3150	−3938
4	−4200	− 5250	− 5250	− 6563	−2540	−3175	−3175	−3969
5	+2600	+ 3250	+ 3250	+ 4063	+1660	+2075	+2075	+2593

Sheet 6/5

+ denotes compression.
− denotes tension.
Stresses given are in pounds.

TOTAL STRESSES FOR ORDINARY PRINCIPALS.
SPANS 30 TO 40 FT.



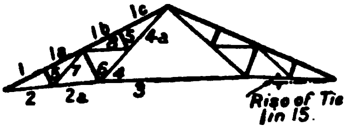
Slope of roof :—
Slated roofs 30° slope.
Sheeted „ 24½° „

Member	Slated Type				Sheeted Type			
	30 ft. Span		40 ft. Span		30 ft. Span		40 ft. Span	
	12′-0″ centres	15′-0″ centres	12′-0″ centres	15′-0″ centres	12′-0″ centres	15′-0″ centres	12′-0″ centres	15′-0″ centres
1	+10200	+12750	+13600	+17000	+8940	+11175	+11920	+14900
1a	+ 8010	+10012	+10680	+13350	+7200	+ 9000	+ 9600	+12000
1b	+ 8880	+11100	+11840	+14800	+8460	+10575	+11280	+14100
2	− 9960	−12450	−13280	−16600	−9000	−11250	−12000	−15000
3	− 4650	− 5813	− 6200	− 7750	−3840	− 4800	− 5120	− 6400
4	− 5100	− 6375	− 6800	− 8500	−5220	− 6525	− 6960	− 8700
5	+ 3000	+ 3750	+ 4000	+ 5000	+2310	+ 2888	+ 3080	+ 3850

Sheet 6/6

+ denotes compression.
— denotes tension.
Stresses given are in pounds.

TOTAL STRESSES FOR ORDINARY PRINCIPALS. SPANS 50 TO 60 FT.



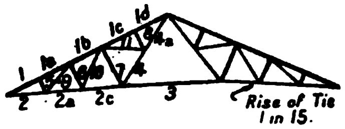
Slope of roof :—
Slated roofs 30° slope.
Sheeted „ 24½° „

Member	Slated Type				Sheeted Type			
	50 ft. Span		60 ft. Span		50 ft. Span		60 ft. Span	
	12′-0″ centres	15′-0″ centres	12′-0″ centres	15′-0″ centres	12′-0″ centres	15′-0″ centres	12′-0″ centres	15′-0″ centres
1	+21000	+26250	+25200	+31500	+15800	+19750	+18960	+23700
1a	+20250	+25312	+24300	+30375	+15600	+19500	+18720	+23400
1b	+19500	+24375	+23400	+29250	+15300	+19125	+18360	+22950
1c	+18650	+23312	+22380	+27974	+15000	+18750	+18000	+22500
2	−20250	−25313	−24300	−30374	−15800	−19750	−18960	−23700
2a	−16600	−20750	−19920	−24900	−12800	−16000	−15360	−19200
3	− 8750	−10938	−10500	−13125	− 6400	− 8000	− 7680	− 9600
4	− 8250	−10312	− 9900	−12374	− 6600	− 8250	− 7920	− 9900
4a	−12000	−15000	−14400	−18000	− 9600	−12000	−11520	−14400
5	+ 3250	+ 4063	+ 3900	+ 4876	+ 2050	+ 2563	+ 2460	+ 3075
6	+ 6500	+ 8125	+ 7800	+ 9750	+ 4100	+ 5125	+ 4920	+ 6150
7	− 3750	− 4688	− 4500	− 5625	− 3000	− 3750	− 3600	− 4500
8	− 3750	− 4688	− 4500	− 5625	− 3000	− 3750	− 3600	− 4500

+ denotes compression.
— denotes tension.
Stresses given are in pounds.

Sheet 6/7

TOTAL STRESSES FOR ORDINARY PRINCIPALS. SPANS 70 TO 80 FT.



Slope of roof :—Slated roofs 30° slope.
Sheeted „ 24½° „

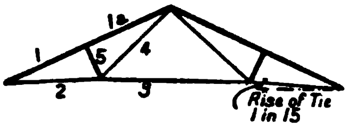
Member	Slated Type				Sheeted Type			
	70 ft. Span		80 ft. Span		70 ft. Span		80 ft. Span	
	12′-0″ centres	15′-0″ centres	12′-0″ centres	15′-0″ centres	12′-0″ centres	15′-0″ centres	12′-0″ centres	15′-0″ centres
1	+29750	+37188	+34000	+42500	+24570	+30713	+28080	+35100
1a	+28910	+36138	+33040	+41300	+24290	+30363	+27760	+34700
1b	+24220	+30275	+27680	+34600	+20860	+26075	+23840	+29800
1c	+22260	+27825	+25440	+31800	+19250	+24063	+22000	+27500
1d	+21350	+26688	+24400	+30500	+19180	+23975	+21920	+27400
2	−28700	−35875	−32800	−41000	−24570	−30713	−28080	−35100
2a	−24640	−30800	−28160	−35200	−21210	−26513	−24240	−30300
2c	−20580	−25725	−23520	−29400	−17850	−22313	−20400	−25500
3	−11900	−14875	−13600	−17000	−10010	−12513	−11440	−14300
4a	−13300	−16625	−15200	−19000	−11270	−14088	−12880	−16100
4	−10150	−12688	−11600	−14500	− 8890	−11113	−10160	−12700
5	+ 3710	+ 4638	+ 4240	+ 5300	+ 2310	+ 2888	+ 2640	+ 3300
6	+ 5600	+ 7000	+ 6400	+ 8000	+ 3500	+ 4375	+ 4000	+ 5000
7	+ 9100	+11375	+10400	+13000	+ 5880	+ 7350	+ 6720	+ 8400
8	+ 3710	+ 4638	+ 4240	+ 5300	+ 2310	+ 2888	+ 2640	+ 3300
9	− 4200	− 5250	− 4800	− 6000	− 3360	− 4200	− 3840	− 4800
10	− 5250	− 6563	− 6000	− 7500	− 4200	− 5250	− 4800	− 6000
11	− 3150	− 3938	− 3600	− 4500	− 2450	− 3063	− 2800	− 3500

+ denotes compression.
— denotes tension.
Stresses given are in pounds.

Sheet 6/8

STANDARD SECTIONS FOR ORDINARY PRINCIPALS. SPANS 20 TO 25 FT.

ROOFS SLATED OR SHEETED.



Slope of roof—Slated roofs 30° slope.
Sheeted „ 24½° „

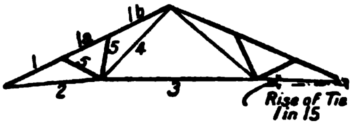
Member	20′-0″ Span		25′-0″ Span	
	12′-0″ centres	15′-0″ centres	12′-0″ centres	15′-0″ centres
	Slated Type			
1	3″ × 3″ × ¼″ ∟ or (3½″ × 3½″ × 5/16″ ∟)	3″ × 3″ × 5/16″ ∟ (3½″ × 3½″ × ⅜″ ∟)	3″ × 3″ × 5/16″ ∟ (3½″ × 3½″ × ⅜″ ∟)	3½″ × 3½″ × 5/16″ ∟ (4″ × 4″ × ⅜″ ∟)
2	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟	2½″ × 2″ × ¼″ ∟	3″ × 2″ × ¼″ ∟
3	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟
4	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟
5	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟
	Sheeted Type			
1	3″ × 3″ × ¼″ ∟ (3½″ × 3½″ × 5/16″ ∟)	3″ × 3″ × ¼″ ∟ (3½″ × 3½″ × 5/16″ ∟)	3″ × 3″ × 5/16″ ∟ (3½″ × 3½″ × ⅜″ ∟)	3″ × 3″ × 5/16″ ∟ (3½″ × 3½″ × ⅜″ ∟)
2	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟
3	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟
4	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟
5	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟

Sheet 6/9

Sections in brackets for member " 1 " allow for flexural stress due to purlin in addition to " principal " stress.

STANDARD SECTIONS FOR ORDINARY PRINCIPALS. SPANS 30 TO 40 FT.

ROOFS SLATED OR SHEETED.



Slope of roof—Slated roofs 30° slope.
Sheeted „ 24½° „

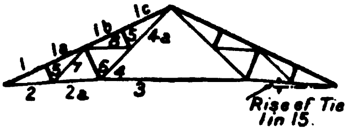
Member	30′-0″ Span		40′-0″ Span	
	12′-0″ centres	15′-0″ centres	12′-0″ centres	15′-0″ centres
Slated Type				
1	3″ × 3″ × 5/16″ ∟ (3½″ × 3½″ × ⅜″ ∟)	3″ × 3″ × 5/16″ ∟ (3½″ × 3½″ × ⅜″ ∟)	2/2½″ × 2″ × ¼″ ⅃∟ (2/3″ × 2½″ × 5/16″ ⅃∟)	2/3″ × 2″ × ¼″ ⅃∟ (2/3″ × 2½″ × 5/16″ ⅃∟)
2	2½″ × 2″ × ¼″ ∟	2½″ × 2″ × ¼″ ∟	3″ × 2″ × ¼″ ∟	3″ × 3″ × ¼″ ∟
3	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟
4	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟
5	3″ × 3″ × ¼″ ∟	3″ × 3″ × ¼″ ∟	2/2½″ × 2″ × ¼″ ⅃∟	2/2½″ × 2″ × ¼″ ⅃∟
Sheeted Type				
1	3″ × 3″ × ¼″ ∟ (3½″ × 3½″ × 5/16″ ∟)	3″ × 3″ × 5/16″ ∟ (3½″ × 3½″ × ⅜″ ∟)	2/2½″ × 2″ × ¼″ ⅃∟ (2/3″ × 2½″ × ¼″ ⅃∟)	2/2½″ × 2″ × ¼″ ⅃∟ (2/3″ × 2½″ × ¼″ ⅃∟)
2	2″ × 2″ × ¼″ ∟	2½″ × 2″ × ¼″ ∟	2½″ × 2″ × ¼″ ∟	3″ × 2″ × ¼″ ∟
3	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟
4	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟	2″ × 2″ × ¼″ ∟
5	2½″ × 2½″ × ¼″ ∟	2½″ × 2½″ × ¼″ ∟	3″ × 3″ × ¼″ ∟	3″ × 3″ × 5/16″ ∟

Sheet 6/10

Sections in brackets for member " 1 " allow for flexural stress due to purlin in addition to " principal " stress.

STANDARD SECTIONS FOR ORDINARY PRINCIPALS. SPANS 50 TO 60 FT.

ROOFS SLATED OR SHEETED.



Slope of roof—Slated roofs 30° slope.
Sheeted „ 24½° „

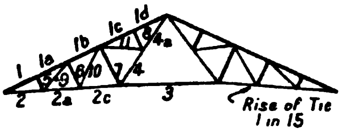
Member	50′-0″ Span		60′-0″ Span	
	12′-0″ centres	15′-0″ centres	12′-0″ centres	15′-0″ centres
	Slated Type			
1	2/3½″×2½″×¼″ ┘└ (2/3½″×2½″×5/16″ ┘└)	2/3½″×2½″×5/16″ ┘└ (2/3½″×2½″×⅜″ ┘└)	2/3½″×2½″×5/16″ ┘└ (2/4″ ×3″ ×⅜″ ┘└)	2/3½″×2½″×⅜″ ┘└ (2/4″ ×3½″×⅜″ ┘└)
2	2/2½″×2″ ×¼″ ┘└	2/3″ ×2″ ×¼″ ┘└	2/2½″×2″ ×¼″ ┘└	2/3″ ×2″ ×¼″ ┘└
3	2/2½″×2″ ×¼″ ┘└	2/2½″×2″ ×¼″ ┘└	2/2½″×2″ ×¼″ ┘└	2/2½″×2″ ×¼″ ┘└
4	1/2½″×2″ ×¼″ └	1/3″ ×2″ ×¼″ └	1/3″ ×2″ ×¼″ └	1/3″ ×2″ ×5/16″ └
5	1/2½″×2½″×¼″ └	1/2½″×2½″×¼″ └	1/3″ ×3″ ×¼″ └	1/3″ ×2″ ×¼″ └
6	2/2½″×2″ ×¼″ ┘└	2/3″ ×2″ ×¼″ ┘└	2/3″ ×2½″×¼″ ┘└	2/3″ ×2½″×¼″ ┘└
7	1/2½″×2″ ×¼″ └	1/2½″×2″ ×¼″ └	1/2½″×2″ ×¼″ └	1/2½″×2″ ×¼″ └
	Sheeted Type			
1	2/2½″×2″ ×¼″ ┘└ (2/3″ ×2½″×¼″ ┘└)	2/3″ ×2″ ×¼″ ┘└ (2/3″ ×2½″×5/16″ ┘└)	2/3½″×2½″×¼″ ┘└ (2/3½″×2½″×5/16″ ┘└)	2/3½″×2½″×5/16″ ┘└ (2/4″ ×3″× 5/16″ ┘└)
2	2/2½″×2″ ×¼″ ┘└	2/2½″×2″ ×¼″ ┘└	2/2½″×2″ ×¼″ ┘└	2/3″ ×2″ ×¼″ ┘└
3	2/2½″×2″ ×¼″ ┘└	2/2½″×2″ ×¼″ ┘└	2/2½″×2″ ×¼″ ┘└	2/2½″×2″ ×¼″ ┘└
4	1/2½″×2″ ×¼″ └	1/2½″×2″ ×¼″ └	1/2½″×2″ ×¼″ └	1/3″ ×2″ ×¼″ └
5	1/2½″×2½″×¼″ └	1/2½″×2½″×¼″ └	1/2½″×2½″×¼″ └	1/2½″×2½″×¼″ └
6	2/2½″×2″ ×¼″ ┘└	2/2½″×2″ ×¼″ ┘└	2/2½″×2″ ×¼″ ┘└	2/2½″×2″ ×¼″ ┘└
7	1/2½″×2″ ×¼″ └	1/2½″×2″ ×¼″ └	1/2½″×2″ ×¼″ └	1/2½″×2″ ×¼″ └

Sheet 6/11

Sections in brackets for member " 1 " allow for flexural stress due to purlin in addition to " principal " stress.

STANDARD SECTIONS FOR ORDINARY PRINCIPALS. SPANS 70 TO 80 FT.

ROOFS SLATED OR SHEETED.



Slope of roof—Slated roofs 30° slope.
Sheeted „ 24½° „

Member	70′-0″ Span		80′-0″ Span	
	12′-0″ centres	15′-0″ centres	12′-0″ centres	15′-0″ centres
Slated Type				
1	2/3½″×2½″×$\frac{5}{16}$″ ⅃∟ (2/4″ ×3″ ×$\frac{5}{16}$″ ⅃∟)	2/3½″×3″ ×$\frac{5}{16}$″ ⅃∟ (2/4″ ×3½″×⅜″ ⅃∟)	2/3½″×3″ ×$\frac{5}{16}$″ ⅃∟ (2/5″ ×3″ ×$\frac{5}{16}$″ ⅃∟)	2/4″ ×3½″×$\frac{5}{16}$″ ⅃∟ (2/6″ ×3″ ×⅜″ ⅃∟)
2	2/3″ ×2″ ×¼″ ⅃∟	2/3″ ×2″ ×$\frac{5}{16}$″ ⅃∟	2/3″ ×2″ ×$\frac{5}{16}$″ ⅃∟	2/3½″×2½″×$\frac{5}{16}$″ ⅃∟
3	2/2½″×2″ ×¼″ ⅃∟	2/3″ ×2″ ×¼″ ⅃∟	2/3″ ×2″ ×¼″ ⅃∟	2/3″ ×2″ ×$\frac{5}{16}$″ ⅃∟
4	1/3″ ×2″ ×¼″ ∟	1/3½″×2½″×¼″ ∟	2/2½″×2″ ×¼″ ⅃∟	2/2½″×2″ ×¼″ ⅃∟
5	1/3″ ×3″ ×¼″ ∟	1/3″ ×3″ ×¼″ ∟	2/2½″×2″ ×¼″ ⅃∟	2/2½″×2″ ×¼″ ⅃∟
6	2/3″ ×2½″×¼″ ⅃∟	2/3″ ×2½″×¼″ ⅃∟	2/3½″×2½″×¼″ ⅃∟	2/3½″×2½″×¼″ ⅃∟
7	2/4½″×3″ ×$\frac{5}{16}$″ ⅃∟	2/4½″×3″ ×$\frac{5}{16}$″ ⅃∟	2/5″ ×3″ ×$\frac{5}{16}$″ ⅃∟	2/5″ ×3″ ×$\frac{5}{16}$″ ⅃∟
8	1/2½″×2″ ×¼″ ∟	1/2½″×2″ ×¼″ ∟	1/2½″×2″ ×¼″ ∟	1/2½″×2″ ×¼″ ∟
Sheeted Type				
1	2/3½″×2½″×¼″ ⅃∟ (2/3½″×2½″×⅜″ ⅃∟)	2/3½″×2½″×$\frac{5}{16}$″ ⅃∟ (2/4″ ×3″ ×$\frac{5}{16}$″ ⅃∟)	2/3½″×2½″×$\frac{5}{16}$″ ⅃∟ (2/4″ ×3″ ×$\frac{5}{16}$″ ⅃∟)	2/3½″×3″ ×$\frac{5}{16}$″ ⅃∟ (2/4″ ×3″ ×⅜″ ⅃∟)
2	2/2½″×2″ ×¼″ ⅃∟	2/3″ ×2″ ×¼″ ⅃∟	2/3″ ×2″ ×¼″ ⅃∟	2/3″ ×2″ ×$\frac{5}{16}$″ ⅃∟
3	2/2½″×2″ ×¼″ ⅃∟	2/2½″×2″ ×¼″ ⅃∟	2/2½″×2″ ×¼″ ⅃∟	2/3″ ×2″ ×¼″ ⅃∟
4	1/2½″×2″ ×¼″ ∟	1/3″ ×2″ ×¼″ ∟	1/3″ ×2″ ×¼″ ∟	1/3″ ×2″ ×$\frac{5}{16}$″ ∟
5	1/2½″×2½″×¼″ ∟	1/2½″×2½″×¼″ ∟	1/3″ ×3″ ×¼″ ∟	1/3″ ×3″ ×$\frac{5}{16}$″ ∟
6	2/2½″×2″ ×¼″ ⅃∟	2/2½″×2″ ×¼″ ⅃∟	2/3″ ×2″ ×¼″ ⅃∟	2/3″ ×2″ ×¼″ ⅃∟
7	2/3½″×3″ ×$\frac{5}{16}$″ ⅃∟	2/3½″×3″ ×$\frac{5}{16}$″ ⅃∟	2/4″ ×3″ ×$\frac{5}{16}$″ ⅃∟	2/4″ ×3″ ×$\frac{5}{16}$″ ⅃∟
8	1/2½″×2″ ×¼″ ∟	1/2½″×2″ ×¼″ ∟	1/2½″×2″ ×¼″ ∟	1/2½″×2″ ×¼″ ∟

Sheet 6/12

Sections in brackets for member " 1 " allow for flexural stress due to purlin in addition to " principal " stress.

Procedure for Calculation of Stresses.—If some other type of principal is desired other than those standardized, then the procedure given hereafter, if followed out systematically, will greatly facilitate the calculation of the stresses.

(1) Draw a line diagram of the roof principal to a suitable scale representing lines drawn through the centres of gravity of the various members.

(2) Calculate the total amounts of wind and dead loads on the principal, using the loads per square foot as given previously, if applicable. The wind load should be equally distributed over one slope—say the left—and the dead load over both slopes of the roof.

(3) Indicate the position, direction, and amount of each external force on the line diagram by means of an arrow. Note that these forces will only act at intersections of braces with rafters, or in the case of ceiling loads, with tiebar.

(4) Number the line diagram, allotting one number to each space between any two external forces, and also one to each internal space. The numbers should start, say, at the left shoe, and progress in numerical order round external forces, reactions, and spaces. Set out the polygon of external forces to suitable scale, beginning with Force 1, and so on in numerical order, noting that each line representing any force must be set off in the direction in which the force acts.

(5) Before the polygon of external forces can be completed it is necessary to determine the amount and direction of each reaction. This problem may be solved graphically by constructing a polar diagram and a funicular polygon, but, as in previous chapters, the reactions have been determined analytically, and as this method is more direct, this procedure will be adopted here, and is as follows :—

(*a*) When both ends of the principals are considered as fixed, the direction of the reactions is parallel to the forces producing them, and the amount transmitted to each support is inversely proportional to the distance of the supports from the point of intersection, of the resultant force, with a horizontal line drawn through the reaction points. This procedure is the most correct, and is the one generally used.

(*b*) When one end is free to move and the other end is fixed, the reaction at the free end will act in a vertical direction only, and therefore the reaction may be found graphically by the rule known as " the concurrence of three forces in equilibrium." Analytically, the vertical reactions for the wind and dead loads may be found as for (*a*). The whole of the horizontal component of the wind force is resisted at the fixed end, so that this resisting force is added to the vertical reaction at this support.

(c) If both ends are considered free to move as for Types 2, 3, and 4 in the preceding chapters, the vertical reactions at each support are found as for (a) for the dead load, and for the vertical component of the wind force, whilst the horizontal component of the wind force is divided equally between the reactions.

(d) To find the reactions for a principal with a load or loads hung to the bottom chord, find the reactions for roof loads as for *a*, *b*, and *c*, then from the reaction point of the polygon of external forces, set off the tiebar loads above and below this point, the amount in either direction being found as for a beam, i.e. the amount of load transmitted to each support being inversely proportional to the distance of the supports from the load or loads.

(6) The location of the reaction point enables the polygon of external forces to be completed, and the stress diagram may now be drawn, commencing at the point where the number of forces does not exceed two—a condition generally existent at the shoe—then follow the numbers in numerical order. The proof of the correctness of the diagram is in the closing of the last line.

(7) The amount of stress in any member may be scaled from the diagram, and the nature of the stress may be found as follows :—

(a) In numerical order, read off from the line diagram the number on each side of the member in question round the joint considered.

(b) Put an arrow on the corresponding line in stress diagram, from 1 to 2, and so on.

(c) Transfer this direction to the member in the line diagram, then, if the arrow points towards the joint, the member is in compression, and if away from the joint, is in tension. Note that the arrow on the line diagram only refers to the joint round which the numbers were taken.

The foregoing procedure will be more easily followed if reference is made to Plate 1/B.

Choice of Sections.—Before choosing the sections for the various members, the factors which decide them, and which are given hereafter, should be considered. Those members subject to a direct compressive stress should have the radius of gyration as large as possible relative to the area.

Those members subject to bending should have the modulus of section as large as possible relative to the area.

The number of different sections in any given structure should be kept to a minimum, without, of course, undue excess of material.

The sections adopted should be those which can be obtained cheaply and quickly.

The sections will be confined generally to joists, channels, angles, tees, and flats.

The prices of these materials are given hereafter, and are those generally adopted throughout the United Kingdom by the various rolling mills. The basis for all except joists is the current price of angles—quoted from time to time—having six to twelve united inches (i.e. the sum of the two flanges) × ¼" thick, upwards. The extras for the various sections are fairly constant relative to the current basis price for angles, the latter, of course, fluctuating from time to time according to the state of the market.

Limits and extras for angles, flats, tees, and channels, with current price of angles as basis :—

		Price per ton.		
		s.	d.	
Angles :—				
6 to 12 united inches × ¼" thick and over,	Basis			
Extras :—				
Under 6 united inches, each inch or part,	Basis	+ 5	0	
Over 12 ,, ,, ,, ,, ,,	,,	+ 5	0	
3½"×2", 3"×2½", 3"×2", 2½"×2½"; ¼" thick or over, ...	,,	+ 5	0	
2½"×2", 2"×2"; ¼" thick or over, .	,,	+ 10	0	
Lengths, 5 ft. and over, without extra.				
Flats :—				
Over 5" wide and upwards, ⅜" thick or over,	Basis	+ 5	0	
Extras :—				
Under ⅜" to $\frac{5}{16}$" thick . . .	Basis	+ 10	0	
,, $\frac{5}{16}$" ,, ¼" ,, under 8" wide	,,	+ 15	0	
,, $\frac{5}{16}$" ,, ¼" ,, 8" wide and over	,,	+ 20	0	
Lengths, 5 ft. to 40 ft., without extra.				
Tees :—				
6 to 10 united inches, $\frac{5}{16}$" thick or over,	Basis	+ 7	6	
Extras :—				
Under 6 united ins., each in. or part	(Basis+7s. 6d.)	+ 5	0	extra.
Over 10 ,, ,, ,, ,, ,,	,,	+ 5	0	,,
Under $\frac{5}{16}$" to ¼" thick . . .	,,	+ 5	0	,,
2½"×2½"×$\frac{5}{16}$", 3"×2½"×$\frac{5}{16}$" . .	Basis	+ 12	6	
2½"×2½"×¼", 3"×2½"×¼", 2"×2"×$\frac{5}{16}$"	,,	+ 17	6	
2½"×2"×¼", 2"×2"×¼" . . .	,,	+ 22	6	
Lengths, 5 ft. and upwards, without extra.				

		Price per ton. s.	d.	
Channels :—				
Web 6 to 12 inches wide . . .	Basis +	5	0	
Extras :—				
Over 12″ to 15″ wide . . .	Basis +	10	0	
Under 6″ wide, each inch or part	(Basis+5s.)+	5	0	extra.

Lengths, 5 ft. and upwards, without extra.

Limits and extras for R.S. Joists, with current basis price of Joists as basis.

R.S. Joists :—

5″×3″ to 14″×6″, excluding 9″×7″ and 10″×8″ In lengths of 5 to 40 ft.,—	Basis (for Joists).

Extras :—			
9″×7″	Basis +	5	0
10″×8″	,, +	10	0
Over 14″×6″ to 16″×6″	,, +	5	0
18″×7″ and 20″×7½″	,, +	10	0
24″×7½″	,, +	20	0
4″×3″ and 4¾″×1¾″	,, +	10	0
Under 4″×3″	,, +	20	0
Lengths over 40 ft., each foot or part . .	,, +	1	0

To illustrate the use of the foregoing table of extras for sections, if the current basis price of angles be £6 per ton, then

			per ton. £	s.	d.
2½″×2″×¼″ angles	will cost	£6+10s.	=6	10	0
2½″×¼″ flats	,, ,,	£6+15s.	=6	15	0
2″×2″×¼″ tees	,, ,,	£6 7s. 6d.+5s.+5s.+5s.	=7	2	6
5″×2½″ channels	,, ,,	£6 5s.+5s.	=6	10	0

To facilitate the delivery of the material from the rolling mills, and to avoid the necessity of obtaining materials from stock (the extra price from rolling mills stock is about 10s. to 15s. per ton, and from merchants' stock anything up to £2 per ton, over basis price), sections should be chosen which are readily obtainable. A list of sections recommended for use is given below :—

Angles :—

2″×2″; 2½″×2″; 2½″×2½″; 3″×2″; 3″×2½″; 3″×3″; 3½″×3″; 3½″×3½″; 4″×3″; 3½″×2½″; 4″×3½″; 4″×4″; 5″×3″; 6″×3″; 6″×4″. Angles may be obtained easily to any thickness stated in the lists supplied by the various manufacturers. Preference should be given to those sections having flanges of equal size.

Tees :—

2″×2″×¼″; 2½″×2½″×5/16″, or ⅜″; 3″×3″×5/16″, or ⅜″; 4″×3″×⅜″; 4″×4″×⅜″, 7/16″ or ½″; 3½″×3½″×⅜″, 7/16″ or ½″; 4″×5″×7/16″, or ½″; 5″×4″×7/16″, or ½″; 6″×3″×⅜″, 7/16″, or ½″; 6″×4″×7/16″, or ½″.

Flats.—Any listed sections.

Joists and Channels.—Any section embodied in the list of the British Standards Committee.

Safe Stresses.—The sections of members subjected to compressive stresses should—to facilitate the design—be determined from the tables given in the various handbooks issued to steel users "gratis" by the various manufacturers, using a factor of safety of 4, and limiting the ratio of the radius of gyration to length to $\frac{1}{160}$, otherwise resort will have to be had to the usual strut formulæ, necessitating cumbersome calculations.

Other parts, i.e. tiebars, plates, rivets and bolts, should be designed to withstand the following safe stresses :—

Tension (steel)	up to	16,800 lbs.	(7½ tons)	per sq. inch.
Shear	„ „	15,680 „	(7 „)	„ „
Bearing	„ „	22,400 „	(10 „)	„ „

Factors which affect design.—A framed structure should be designed so that the external forces which may be applied to it will be safely resisted with the minimum use of materials, not only by the parts as units, but also by the units as a whole.

The theory of structures should be used as a means only to the end desired. That is to say, that theory and practice must be mutually applied so that economy as well as stability is obtained; a structure designed with a minimum of material need not necessarily be the cheapest.

Members of a framed structure should take up, as nearly as possible, a position whereby the centre of gravity (the centre of resistance) coincides with the centre of the force it is intended to resist. Applying this factor to a roof principal, the centre of gravity of each member should coincide with the line diagram of the principal, and the point of intersection of the rafter and the main tie, at the shoe, should also be intersected by a line drawn vertically through the centre of gravity of the support.

Rivets should be arranged so as to be centric with the forces they are intended to resist—as far as practicable.

Smithwork, complicated lattice work, complicated connections, etc., are expensive, and should generally be avoided. Plate connections should be arranged and shaped, so as to reduce the amount of cutting and of waste material to a minimum.

Parts should, as far as practicable, be made interchangeable.

The diameter of all holes in any complete portion of framework should be kept the same, as far as possible.

Preference should generally be given to rigid sections for the members of any structure, even if subjected to tensional stress only.

Ends of tension members should generally be left square—fitting is usually unnecessary, and therefore, of course, an economic waste of labour.

Materials for foreign shipment, if provided with large projections, incur extra cost if freight is charged by measurement, as is generally the case for such parts, and it is often advisable for economy, and to prevent damage, to avoid such projections as may be attached at the site.

Foundation bolts should have square necks, and consequently square heads.

Connections should be simplified and also repeated as far as practicable.

The connections of all frames affected by flexural stresses should be made by means of rivets or of close-fitting bolts.

Where girder flanges are made up of angles and plates, the angles should have, as far as practicable, 50% of the total area of the flange.

Joints of girders or framework should be arranged so as to facilitate delivery and erection.

Girders and stancheons should be suitably stiffened in a lateral direction, especially at joints, or the safe stress reduced according to the conditions.

The span for joists should generally be limited to twenty times the depth, for plate girders to sixteen times the depth, and for braced girders to twelve times the depth.

The thickness of any plate or bar should not be less than $\frac{1}{4}''$; $\frac{5}{16}''$ is preferable as a limit in many situations.

Butting surfaces of members subject to compression should be perfectly fitted.

Steelwork, before leaving manufacturer's yard, should have a coat of oil, which should be scraped off before the final coat of paint is applied. Steelwork embedded in concrete should be coated with Portland cement-wash, applied after a thorough cleansing, in preference to paint.

DESCRIPTION OF DESIGN FOR SHED. TYPE 3.

Plates 6/L and 6/M illustrate the general arrangement of the framework, also the details of the connections, for the shed (Type 3), the sections for which were determined in Chapter I. The special features for the various details are as follows :—

Side Stancheon Base, Plate 6/L.—The forces acting on the base are the lateral reaction (4777 lbs.), and the vertical reaction (8647 lbs.). The minimum area of the base should be such that the pressure on the concrete, due to the vertical reaction, does not exceed 12 to 15 tons per square foot. The area of the anchor bolts, of the rivets connecting the baseplate to the base angles, and of the rivets connecting the base angles to the stancheon shaft, should safely resist the lateral reaction; the latter rivets also resist a portion of the vertical reaction. The anchor bolts are situated as near as possible to the centre of the base to allow the latter to act approximately as a pin joint.

Shoe Connections H and J, Plate 6/L.—Lines drawn through the centre of gravity of the various members are arranged to intersect at a point.

The rivets connecting the rafter and main tie to the shoe plate are designed to resist the total stresses in these members.

The rivets connecting the shoe plate to the shoe angles, the latter to the stancheon cap plate, the cap plate to the angle cleats, and the angle cleats to the stancheon shaft, are designed to resist the force necessary to equalize—about the stay connection—the moment of the shearing force. This force will equal $(4777-1800\times\frac{15}{3})=8931$ lbs. If the top rail reacts against the stancheon, then this reaction (600 lbs.) should be added to 8931 lbs. to give the total force.

Strut Connections B, D, F, and H, Plate 6/L.—The rivets connecting the strut to the rafter plate resist the compressive stress in the strut, whilst the rivets connecting the plate to the rafter resist the difference in the compressive stresses in the portions of the rafter on each side of the connection, plus the stress in the strut.

Strut and Tie Connections C and G, Plate 6/L.—The rivets connecting the ties and the strut to the rafter plate resist the stresses in these members, whilst those connecting the plate to the rafter act as for plate in connection B. If the rafters are jointed at this connection, as is often necessary, then the rivets connecting the rafters to the plates should resist the total stresses in the rafters, and not their difference.

Connection E, Plate 6/L.—The rivets connecting the rafter, suspender and diagonal braces to the plate resist the stresses in these members.

Connections L and Q, Plate 6/L.—The rivets connecting the ends of the members to the plate resist the stresses in these members, whilst those connecting the plate to the main tiebar resist the difference in the stresses in the portions of the tiebar adjacent to the connection.

Connections M and P, Plate 6/L.—The rivets connecting the members to the plates resist the stresses in these members.

Connections S and T, Plate 6/L.—The rivets connecting the ends of the strut and tie to the plate resist the stresses in these members, whilst those connecting the plate to the main diagonal member resist the difference in the stresses of the portions of the diagonal on each side the connection.

Connections K and R, Plate 6/L.—The rivets connecting the stay to the web of the ⊢ cleat resist the stress in the stay, whilst the bolts connecting the flange of the cleat to the stancheon resist the lateral and vertical components of the diagonal force in the stay. Bolts are used in this instance, as the lateral component of the force in the windward stay is of a tensile nature, and rivets should not be used where the shank is subject to a tensile stress.

With regard to the arrangement of the rivets in the foregoing connections, the distance from the end of a bar to the centre of the first rivet should not be less than one and a half times the diameter of the rivet, and preferably also not less than five-eighths the width of the bar. The diameter of the rivet should not exceed one-third the width of the bar; and the centres of the rivets should not be less than three times their diameter, nor more—for compression members—than 6″ or sixteen times the thickness of the thinnest outside member or plate measured in the line of stress, and forty times if measured at right angles to the line of stress. Rivets at the ends of compression members should not be spaced further apart than four times the diameter of the rivet.

Detail A, Plate 6/M.—The rivets connecting the web of the joist rail to the angle cleats resist the wind pressure acting on the rail, as also do the rivets connecting the cleats to the stancheon, plus the weight of the rail and side sheets. This connection should also be made strong enough to enable the rails to stiffen the stancheons in the direction of the least radius of gyration.

Detail C, Plate 6/M.—The rivets connecting the horizontal ⌉⌈ bar to the cleat resist the stress in the bar, i.e. 9844 lbs. The rivets connecting the flat bar to the cleat, and the cleat to the stancheon, resist the diagonal and vertical components, respectively, of the force 9844 lbs.

Detail D, Plate 6/M.—The rivets connecting the flat bar to the cleat, and the cleat to the stancheon, resist the forces described for Detail C.

Detail E, Plate 6/M.—The rivets connecting the horizontal ⌉⌈ bar to the cleat, and the bolts connecting the cleat to the stancheon, resist the stress in the bar, i.e. 9844 lbs.

Detail F, Plate 6/M.—The rivets connecting the angle bracing to the plate resist the stress in the brace, and the rivets connecting the plate to the channel rafter resist the components of the force in the brace, the amount of the components being obtained from the diagonal force by means of the "triangle of forces."

Detail G, Plate 6/M.—The rivets connecting the channel eaves bar to the angle cleats, and the angle cleats to the stancheon, resist the reaction at the end of the channel due to the vertical load and to the side-wind force. The rivets connecting the rafter channel to the plates resist the stresses in the rafter, and also the reaction of the triangular girder of which the rafter forms the flange. The rivets connecting the gable horizontal ⌉⌈ bar to the plate resist the stress in the bar, i.e. 12,305 lbs. The rivets connecting the plates to the stancheon resist the stresses in the members to which they connect, i.e. the channel rafter, and the double angle strut.

Detail H, Plate 6/M.—The rivets connecting the various members to the plate resist the forces in these members, whilst the rivets connecting the plate to the principal simply retain these bars in position vertically, and in addition add to the rigidity of the framework at this point.

Detail J, Plate 6/M.—The rivets connecting the double angle strut to the cleats, and the latter to the apex of the principal, resist the stresses in the strut. The rivets connecting the diagonal bar to the plate resist the stress in the bar, whilst the rivets connecting the plates to the principal rafter resist the components of the diagonal forces, such components being found from the diagonal force by the "triangle of forces."

General Note.—Connections at which the external forces are applied should be so designed that the rivets connecting the plate or member, acting as a support for these external loads, resist these forces in addition to the internal stresses in the members.

CHAPTER VII.

EXTERNAL COVERINGS.

Galvanized Steel Sheets.—Galvanized steel sheets as used for the external covering for sheds are usually corrugated, and are of two kinds, i.e. the "Italian" or "Canadian" kind having a corrugation at each side, and one in the middle, running the full length of the sheet, and the common type of corrugated sheet having corrugations at 3″ to 5″ pitch across the width, and running the full length of the sheet.

The usual thicknesses for galvanized steel sheets are No. 16 to No. 26 gauge (Birmingham Wire Gauge). Nos. 24 and 26 gauge sheets are only suitable for temporary work in this country, or for use in tropical or semi-tropical countries, Nos. 20 and 22 gauge sheets for use in temporary or otherwise unimportant buildings, and No. 20 to No. 16 gauge sheets for better class work in this country.

The "Canadian" type is very little used. Ordinary corrugated sheets are stocked by manufacturers and merchants in lengths of from 4 ft. to 8 ft., the widths of 2′-0″ and 2′-6″, and gauges No. 20 to No. 24. Sheets of other dimensions than these are charged for at an extra price per ton.

It should be noted that although a sheet may be stated as 2′-0″ or 2′-6″ wide, this dimension represents the net width when laid with one corrugation side lap, and that the sheet itself is really about 2″ to 2½″ wider than the above dimensions, also that the width is usually specified—especially for gauges from No. 18 to No. 24—in terms of the number and pitch of the corrugations. A 2′-0″ wide sheet having corrugations at 3″ pitch is described as an 8/3″ sheet, and a 2′-6″ sheet as a 10/3″ sheet.

The pitch of the corrugations should always be specified, as, although usually sheets are understood to have corrugations at 3″ pitch, the heavier gauges, e.g. No. 18, and especially No. 16, are sometimes manufactured with 5″ pitch corrugations. A sheet with 3″ corrugations is usually ¾″ deep, and with 5″ corrugations 1¼″ deep.

The zinc coating is usually thin unless specially ordered otherwise, and for important work the amount deposited on each side should be specified as from 2½ to 4 ozs. per square foot, according to the particular requirements. It is usual in this country to use a slightly heavier gauge of sheets for the roof than for the walls.

Sheets may be laid with a single corrugation side lap and a 6-inch end lap, but for good work, and especially for roofs, the side laps should be increased to two corrugation side laps, and the end lap to 9 inches, the slope of the roof being not less than 24½°. In arranging the end laps of the sheets, the laps should be arranged to break joint, otherwise—owing to the ends of the sheets being frequently out of square—an irregular line of joints is created which gives a very bad appearance to the structure.

The supports are usually spaced at from 6 ft. to 8 ft. centres, according to the thickness, and are of angle, tee, channel, joist or other stiff section.

Sheets are secured to the steel supports (termed "rails" for the sides and gables, and "purlins" for the roof) by means of galvanized hookbolts or by specially constructed clips and ordinary bolts. The attachments should be spaced one at each corner and one at the centre of the sheet, and for good work each bolt should have one large, curved, diamond-shaped galvanized washer, and one lead washer, or alternatively a patent washer known as the "limpet" washer. If the sheets are to be attached to timber rails or purlins, then 2½″ × 5⁄16″ mushroom-head galvanized driving screws should be used, with washers similar to those for hookbolts, the positions of the driving screws being similar to those for the hookbolts.

The connection of sheets to sheets may be made by either galvanized bolts or rivets, and it should be noted that the holes for these connections should be punched from the inside. The sizes of bolts in common use are :—

For two thicknesses, ¾″ long × ¼″ dia.
For four thicknesses, 1″ long × ¼″ dia.

For rivets use :—

For two thicknesses, ⅜″ to 7⁄16″ long × ¼″ dia.
For four thicknesses, ½″ to ⅝″ long × ¼″ dia.

Sheet bolts should have one circular galvanized washer under the head, and one lead washer (or preferably one "limpet" washer) under the nut. Rivets should have one circular washer under the head formed by the erector.

Note that where "limpet" washers are used in preference to ordinary washers all bolts should be ¼″ longer.

Sheet 7/1 illustrates the various arrangements of laps and bolts, and Sheet 7/2 the various methods of attachment, together with the number of bolts or rivets required for the laps. Rivets and bolts should always be spaced in the ridges of the corrugations, more particularly on roofs. To ensure a perfectly dust-proof joint, the edges of the sheets may with advantage be painted with white lead paint, or, alternatively, strips of canvas soaked in white lead paint may be inserted between the joints.

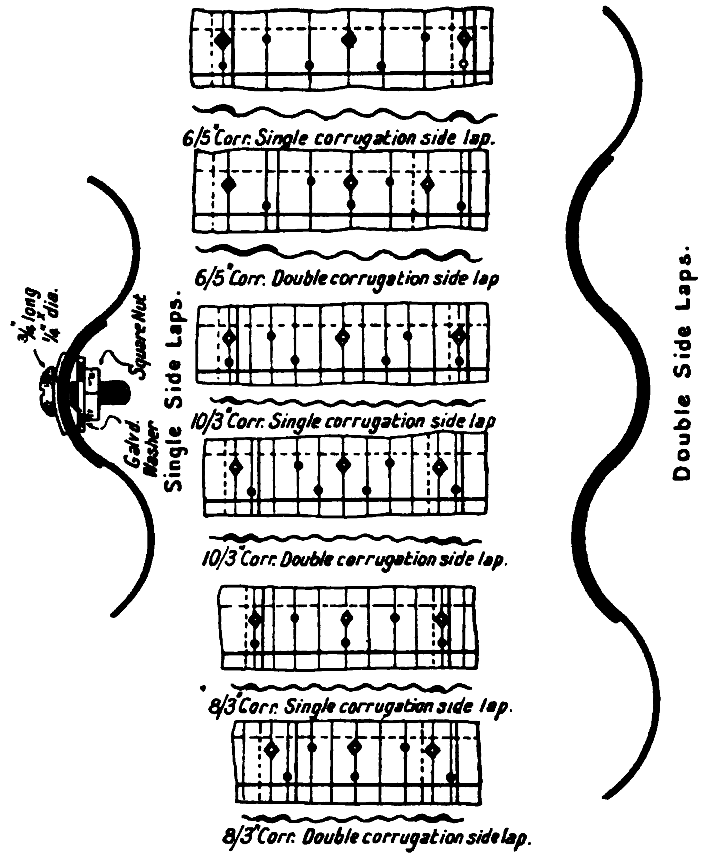
The life of corrugated sheets may—according to the state of the atmosphere to which they are exposed—extend from 10 to 15 years by the judicious application of paint, but before applying this preservative the sheets should be exposed to the oxidizing effect of the atmosphere for about three months, to allow them to come to a suitable condition to receive the paint, or alternatively, a similar effect may be obtained by applying a priming coat of a composition compounded of equal parts (one each) of chloride of copper, nitrate of copper, and sal-ammoniac, dissolved in 64 parts of water, to which should be afterwards added one part hydrochloric acid. This composition turns the sheets black and leaves them ready for paint within twenty-four hours of the application. If the first coat be of oxide of iron paint, the destructive effect of a humid or acid-laden atmosphere will be reduced to a minimum.

Where condensation is likely to take place—a frequent fault with sheeted roofs—an absorbent lining of asbestos paper, millboard, or other absorbent and neutralizing material should be provided with ample attachments to the sheets or framework.

The weight per square of 100 feet of galvanized corrugated sheets is given on Sheet No. 7/3.

Galvanized Steel Ridging.—Ridging should be provided at the apex of the roof to cover the space between the adjacent roof sheets, or alternatively, the sheets on one slope may be curved (the minimum radius for curving sheets is 12″) at the ends, and lapped over the sheets on the other slope. The profile of the ridging generally takes the form shown on Plate 6/L, the wings being sometimes provided with small flanges at the sides. The wings may be of flat sheet, or arranged with corrugations to suit the sheets, in which case the flanged sides are dispensed with. The thickness of the ridging should be slightly greater than that of the sheets, to which it is secured by means of 1″ × ¼″ galvanized bolts, spaced at 9″ centres, and provided with washers similar to those described for connecting the sheets.

ARRANGEMENT OF SIDE AND END LAPS WITH BOLTS FOR CORRUGATED SHEETS.


6/5" Corr. Single corrugation side lap.
6/5" Corr. Double corrugation side lap
10/3" Corr. Single corrugation side lap
10/3" Corr. Double corrugation side lap.
8/3" Corr. Single corrugation side lap.
8/3" Corr. Double corrugation side lap.
3/4" long 1/4" dia.
Square Nut
Galvd. Washer
Single Side Laps.
Double Side Laps.

Sheet 7/1

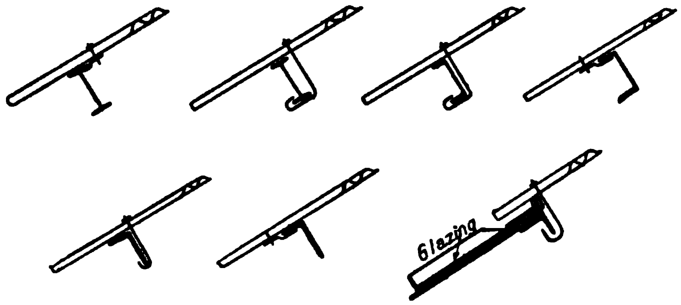
PARTICULARS OF FASTENINGS FOR CORRUGATED SHEETS.

NUMBER OF RIVETS OR BOLTS IN LAPS.

Length of Sheet	Side Laps		End Laps			
	Single Corrugations	Double Corrugations	With Single or Double Side Laps		Single Side Laps	Double Side Laps
			8/3″ Corrs.	10/3″ Corrs.	6/5″ Corrs.	6/5″ Corrs.
4′-6″ and 5′-0″	3	5	3	4	4	3
5′-6″ and 6′-0″	4	6	3	4	4	3
6′-6″ and 7′-0″	5	8	3	4	4	3
7′-6″ and 8′-0″	6	10	3	4	4	3
8′-0″ and 9′-0″	7	12	3	4	4	3

LENGTH OF HOOKBOLTS.

Size of Angle bar	2″	2½″	3″	3½″	4″	4½″	5″	5½″	6″
Length of hookbolt, 3″ corrs.	3¼″	3¾″	4¼″	4¾″	5¼″	5¾″	6¼″	6¾″	7¼″
,, ,, ,, 5″ corrs.	3¾″	4¼″	4¾″	5¼″	5¾″	6¼″	6¾″	7¼″	7¾″



METHODS FOR ATTACHING SHEETS TO PURLINS AND RAILS.

SHEET 7/2

WEIGHT IN LBS. PER SQUARE OF 100 SUPER. FT. OF GALVANIZED AND CORRUGATED STEEL SHEETS.

B.W.G.	6/4″ or 8/3″	7/4″	6/5″ or 10/3″	8/4″
16	347	344	333	339
18	270	268	258	265
20	215	213	207	210
22	180		176	
24	147		143	
26	129		125	

The weights given include for one corrugation side lap and 6″ end laps.

Sheet 7/3

Galvanized Steel Flashings.—Galvanized steel flashings are cheaper than, but inferior to, lead flashings, and should be used—although lead is preferable—at all points where an otherwise waterproof joint is impossible, such as occur at the hips and valleys of roofs, around windows, over outside doors, etc. When used over doors they are often termed "hoods," and if under windows are termed "sills." The thickness should be slightly greater than for the adjacent sheets. Flashings may be used at the corners of the buildings, and to the joints of doorways, if a watertight joint is unobtainable by turning and lapping the sheets.

External Coverings.—The materials, other than corrugated steel sheets, commonly used in the construction of filling in walls for sheds and other similar buildings, are timber boards, brickwork, asbestos cement slabs, and concrete (either plain or reinforced). For roofs, the alternative coverings to steel sheets are roofing felt, natural slates, asbestos cement slates and slabs, tiles, or concrete (either plain or reinforced) covered with asphalt.

Timber Boards.—Spruce or pitch pine boards, ¾" to 1¼" thick according to the span between the supports, are used for the walls of cheap temporary buildings. The boards should be well seasoned before using, otherwise they are liable to split and the joints to open. The joints should be tongued and grooved, and also, if required, may be beaded. The boards should be supported by means of timber rails, or timber fillets secured to steel rails, at distances of from 4 ft. to 6 ft. apart, the brads or nails securing the boards to the supports being of a length equal to about twice the thickness of the boards. If not covered on the outside with felt, then the boards should be preserved by the application of a suitable damp-resisting composition.

Brickwork.—The class of bricks generally used is that known as "stocks," and for good work bricks specially selected from this class should be used, as they are more even in colour than those taken at random. The bricks may be laid with lime or cement mortar, the latter being used only where the unsupported area is greater than that allowable for lime mortar-jointed walls. The usual thickness of filling in walls is one brick (9"), and the unsupported area should be restricted to about 240 square feet, or the thickness of the wall increased, or alternatively reinforced along the horizontal joints, at certain courses, with wire meshing, which is specially manufactured for this purpose. All brick walls should have a damp-proof course immediately above the base and underneath the lowest floor. The walls should be erected between the flanges of the steel framework, which should be formed of 12" deep joists or channels; this arrangement will greatly increase the resistance of the steelwork to compressive stresses. If the depth of the steelwork is, for economic reasons, less than 12", then one-half the total thickness of the wall should be laid between the flanges of the steel supports, the inner half being arranged to cover the inner face of the steel. The monotonous appearance of the plain wall may be relieved by forming piers around the outer flanges of the steel upright, and also by corbelling out two or three courses immediately underneath the eaves gutter. These overhanging courses will form a seat for the gutter, and together with the gutter, if suitably designed, will form a pleasing cornice.

Asbestos Cement Slabs and Slates.—These slabs and slates, as the name implies, are manufactured from a mixture of cement and asbestos fibre. They are fireproof and capable of resisting the deteriorating effect of the atmosphere. They are very strong and light, and consequently reduce the dead weight of the covering, and also the size of the supporting framework. The composite material is sufficiently elastic to resist tensile and compressive stresses, is therefore suitable for resisting bending stresses, and consequently a continuous backing is not required as for slates. The material is easily holed and cut to any shape required, and nails may be driven through without breaking the slabs or slates. The manufacturers of these specialities claim that, as compared with slates, they are lighter, stronger, have better insulating properties, and the initial cost and maintenance cost are less.

Asbestos cement slates are obtainable from stock in various shapes, sizes, and tints. They are usually laid on roof felt and boards, and make a very efficient and effective covering, especially if laid diagonally. The monotonous appearance of the covering may be broken up by occasionally fixing two or three courses of specially shaped slates between the courses of the common type. Although this type of slate may be secured direct to timber or steel laths, and roof boards dispensed with, the efficiency of this procedure is doubtful. The pitch of the roof for asbestos cement slates should not be less than one in three.

Asbestos cement slates are obtainable either plain or corrugated, are more expensive (in first cost) than corrugated steel sheets, but are much more durable, therefore in many situations preferable. Plain slabs are usually nailed to timber studding or to timber boards. The vertical joints, especially if not backed by timber studding, should be made by securing a strip of asbestos cement over the butting edges of the adjacent slabs. The horizontal joints are lap-jointed.

Asbestos cement slabs are obtainable in a corrugated pattern somewhat similar to corrugated steel sheets, the standard length being 8 ft. They may be used with advantage in preference to steel sheets, but the pitch of the roof should not be less than one in four. They are laid on either timber or steel supports, at not more than 30 inches apart, to which they may be secured by means of driving screws to timber, and by clips and bolts or by twisted wire to steel supports. Lead washers should be used on the outer surface to cover the holes.

When laid on roofs, the slabs should have two corrugations side lap and 6" end lap, and for walls with 4" end lap and two corrugations side lap, although the latter lap may be reduced to one corrugation if the side laps are bolted between the supports.

For further information regarding asbestos cement specialities reference should be made to the various trade catalogues.

Natural Slates.—Natural slates may be obtained of various sizes and colours, the latter varying chiefly according to the proportion of iron they contain. The sizes in common use for shed roofs are 24" × 12" (Duchess) and 20" × 10" (Countess). The gauge to which they are laid is calculated by subtracting the lap—generally 3"—from the length, and halving the remainder; the gauge for "Duchess" slates will therefore be 10½", and for "Countess" slates will be 8½". The districts from which slates used in England are generally obtained are North Wales and Westmoreland. There are four qualities, i.e. "bests," "seconds," "thirds," and "tons," the first three qualities being sold by count (1200 to the thousand) and the fourth by weight.

The cost and weight vary according to the qualities, "bests" being dearer and thinner than "seconds," "seconds" dearer and thinner than "thirds," and so on; but the greater initial cost of the better qualities may be neutralized by the greater cost of transport of the inferior and heavier qualities.

Welsh slates may be of a blue, grey, or purple colour, and are cut to uniform sizes. Westmoreland slates are of a greenish colour and vary in size; they are usually heavier than Welsh slates. The slope of a slated roof may vary from 24½° upwards, according to the quality, size, and thickness of the slates used. Large slates, such as the Westmoreland type, are suitable for low-pitched roofs, and small slates of good quality for steep roofs. Slates may be secured at the head or waist—two copper or galvanized iron nails to each slate—and laid direct to timber battens or steel angle laths, but for a good job, and especially in exposed situations, they should be laid on felt and boards. The boards should be unwrought on the surface in contact with the felt, and the felt should be laid with the seams running up the slope.

Slating is commenced from the bottom, and a tilting fillet should be provided at this point, the bottom course of slates being a double one, and the bottom layer being of smaller size than the others. The courses, except where variable sizes of slates are used, will be laid to a constant gauge, which may with advantage be arranged so that the top course may work in without cutting. The top course will be laid double, somewhat similar to the arrangement at the bottom.

Slates may be laid direct upon steel angle laths spaced at centres corresponding to the gauge. Where this arrangement is adopted the size of the slates should be 24" × 12", therefore the laths should be spaced at $10\frac{1}{2}''$ centres. The sizes of the laths vary according to the span, the usual practice being as follows :—

For principals up to 6 ft. centres use $1\frac{1}{4}'' \times 1\frac{1}{4}'' \times \frac{1}{4}''$ ∟ laths.
" " 6 ft. to 8 ft. " " $1\frac{1}{2}'' \times 1\frac{1}{2}'' \times \frac{1}{4}''$ ∟ "
" " 8 ft. " 10 ft. " " $1\frac{3}{4}'' \times 1\frac{3}{4}'' \times \frac{1}{4}''$ ∟ "
" " 10 ft. " 12 ft. " " $2'' \times 2'' \times \frac{1}{4}''$ ∟ "

The position of the bottom lath should be carefully calculated, otherwise the whole system of spacing will be upset. The slates may be secured to the angles by copper, lead, galvanized or composite nails of a length sufficient to clip the lath, or alternatively be secured by means of twisted wire.

Slates laid on battens or laths may be bedded and pointed with mortar on the underside, so as to make the roof covering proof against drifting snow.

An economical system for slating roofs, which also forms a convenient method of ventilating buildings subject to noxious fumes, is that known as the "open slating" system, where the slates are laid with a space of 2" to 3" between the sides, this space being well covered by the slate lapping the joint. This arrangement, although rainproof, is not proof against drifting snow.

Roof Tiles.—Tiles are very little used, owing to their excessive weight, except where more easily obtainable than asbestos cement or natural slates. The types in common use are plain tiles, pantiles, Roman tiles and French tiles, and they are laid on battens or laths similar to ordinary slates, the strength of the supports being increased to suit the heavier covering.

Roofing Felt.—Ordinary roofing felt is used as an external covering for walls and roofs of cheap temporary buildings, but chiefly as an insulating and waterproof material between slates and boards. It is obtainable from stock in rolls of 25 yds. × 32" wide, and should be laid —if used with slates—with the seams running up the slopes of the roofs; the thickness is about $\frac{3}{16}''$. There are many kinds of specially manufactured roof felts, particulars of which may be obtained from the trade catalogues.

Concrete Walls.—Concrete 2" to 3" thick, according to the centres of the supports, if reinforced with steel, makes a good external covering for shed walls, and also for the roof if a layer of asphalt or other similar material is added. The reinforcement may consist of $\frac{5}{16}''$ to $\frac{1}{2}''$ vertical rods, spaced about 12" apart, with an occasional horizontal rod of similar diameter, the rods being secured at the ends to the supporting steelwork. An alternative reinforcement is expanded metal or galvanized wire meshing. There are several patent meshings which have sufficient rigidity, between the supports, to allow of the concrete being applied as for plaster; it is claimed that the use of these meshings dispense with timber shuttering.

Concrete blocks—made on the site—may be used for the walls, if there is sufficient quantity required to justify the outlay upon the plant. Hollow blocks should be used in preference to solid blocks, as they are more economical, lighter, and better non-conductors of heat and cold. Owing to the size of the blocks being considerably larger than bricks, they are cheaper to fix than the latter. They may be used for large unsupported areas, as steel rods or bars may be inserted between the joints.

CHAPTER VIII.

GUTTERS, PIPES, VENTILATION, DOORS, WINDOWS, SKYLIGHTS.

GUTTERS AND RAIN-WATER PIPES.

Material.—Gutters and downpipes are usually made of galvanized iron or steel, or cast iron, the former materials being used for buildings and sheds of a somewhat temporary nature, for buildings and sheds of cheap construction, or buildings and sheds manufactured for shipment abroad. Cast iron is used for better class constructions at home, and sometimes for abroad, although owing to the possibility of excessive breakages its use for foreign shipment is very little favoured.

GUTTERS.

Thickness.—Galvanized gutters are made in thicknesses varying from 22s to 16s gauge, according to the gauge of the sheets used for covering the building, the gauge of the gutters being slightly thicker than that of the sheets. Care should be taken to obtain galvanized gutters with edges stiffened by means of a thin rod enclosed along the edge, or by flanging or curving the edges, or other similar devices.

The thickness of C.I. gutters is dependent upon the size and the distance apart of the supports. The usual thicknesses—for gutters supported every 3 ft.—are :—

6″ to. 8″ diameter gutters, thickness $\frac{1}{8}''$ to $\frac{3}{16}''$.
8″ ,, 10″ ,, ,, ,, $\frac{3}{16}''$,, $\frac{1}{4}''$.
10″ ,, 12″ ,, ,, ,, $\frac{1}{4}''$,, $\frac{5}{16}''$.
12″ ,, 15″ ,, ,, ,, $\frac{5}{16}''$,, $\frac{3}{8}''$.
15″ ,, 18″ ,, ,, ,, $\frac{5}{16}''$,, $\frac{1}{2}''$.

Lengths.—Gutters are usually supplied in 6′-0″ lengths from stock. Longer lengths may be obtained, especially for the larger sizes, but owing to the tendency to warp in casting and to the increased liability to break in handling, the 6′-0″ length is almost universally used.

Sections.—Almost any section of gutter may be obtained quickly and economically, but as makers have standard types and patterns these should—for economy and to facilitate delivery—be used where possible. Eaves gutters are usually of the half-round or ogee type—plain or ornamental—boundary wall and valley gutters of box or semi-box type.

Supports.—Supports are usually placed at about 3′-0″ centres—one at each joint and one at the centre of each gutter—and, of course, should be attached to rigid supports. If the gutter is intended to act as a path along the roof—a frequent necessity for boundary and valley gutters—then the width should be sufficient for this purpose and the thickness increased if necessary, or alternatively a continuous rigid support should be provided for the gutter, the latter arrangement being a necessity for galvanized gutters.

Expansion.—In long stretches of gutter, especially if the joints are flanged, it is advisable to provide expansion joints, which should be suitably covered to prevent leakage.

Capacity.—The capacity should be sufficient to efficiently carry away the rain-water during a heavy rainstorm, and the maximum rate of rainfall is usually assumed as ·12 inch per minute.

The required size of gutter, to suit a given area of gathering surface, may be approximately determined by assuming the discharge as equal to one-half that for a circular pipe of the same diameter, or width, as the gutter.

Assuming the gutter is laid with a fall, towards the outlets, of one-fourth of an inch in every 10 feet, then, according to Kütter's formula, the discharge from half-round gutters will be as follows :—

Diameter.	Area in sq. inches.	Discharge in cu. ft. per min.
6″	14	6·4.
8″	25	14·3
9″	31	20
10″	39	27
12″	56	45
15″	88	83
18″	127	136

If the fall is greater than that adopted above, the discharge will be increased 40% for every ⅛″ up to 1″ in 10 feet.

From the foregoing data, and allowing a factor of safety against overflowing of 25%, the minimum sizes of the gutters, together with the maximum centres of the downpipes, for either the eaves or valleys of pitched roofs will be as follows :—

Eaves, Gutters and Pipes

Dia. of Gutter	Allowable area of roof	Area of Gutter	Span of Roof in feet							
			30	40	50	60	70	80	90	100
6	480 sq. ft.	14 sq. ins.	32	25	20					
8	1070 ,,	25 ,,	70	50	40	35	30	25		
9	1500 ,,	31 ,,	100	75	60	50	40	35	30	
10	2025 ,,	39 ,,	135	100	80	70	60	50	45	40
Valley Gutters and Pipes										
10	2025 sq. ft.	39 sq. ins.	65	50	40					
12	3375 ,,	56 ,,	110	85	65	55	45			
15	6225 ,,	88 ,,		150	120	100	85	75	65	60

Maximum Centres in feet of R.W. Pipes.

Gutters having a different section from the above half-round, should have a similar area and approximate to the same shape.

DOWNPIPES.

The thickness, shape, and general design are generally left to the manufacturer, from whose catalogue a suitable choice of stock design may be made. Pipes are usually supplied in lengths of 6′-0″ and are provided with ears or lugs, if desired, for the purpose of attachment to the support.

Capacity.—The formula before referred to gives the discharge for vertical pipes as follows :—

Diameter.		Discharge in cu. ft. per min.
3	..	42
4	..	95
6	..	297

Owing to the nature of the outlets from gutter to pipe, and from pipe to gully, the discharge given above should be decreased by 50% and the allowable area of gathering surface may be as follows :—

Diameter.		Roof area.
3	..	2100 sq. ft.
4	..	4750 ,,
6	..	9500 ,,

From a comparison of the allowable area of gathering surface for gutters and pipes, the relative sizes of gutters and pipes may be as follows :—

If cesspools are provided to gutter or enlarged heads to the pipes, then for—

Gutters up to 9" diameter use 3" pipes.
„ 10" „ 12" „ „ 4" „
„ 12" „ 15" „ „ 6" „

If no cesspools or enlarged heads are provided, then for—

Gutters up to 8" diameter use 3" pipes.
„ 8" „ 12" „ „ 4" „
„ 12" „ 15" „ „ 6" „

PRACTICAL RULES.

The specification of the American Bridge Company states that the sizes of gutters and downpipes, with gutters laid to a fall of 1 in 15, should be as follows :—

Span of Roof.	Size of Gutter.	Size of Downpipe.
Up to 50 ft. ..	6" ..	4" at 40 ft. centres.
50 „ 70 „ ..	7" ..	5" „ „
70 „ 100 „ ..	8" ..	5" „ „

The English practice as stated by several authors is as follows : Rain-water pipes should have a bore of at least one square inch to every 60 square feet of roof surface. They should be placed at not more than 20 feet centres, and should have gutters not less in width than twice the diameter of the pipe. For countries subject to heavy downfalls the diameter of the pipes, and therefore the sizes of the gutters, should be increased by about 100 %.

The American practice is to provide downpipes having 80 % of the capacity stated above for English practice.

Practice Recommended.—Comparing the results found by Kütter's formula with the practical rules given, the sizes of the gutters found compare favourably with the practice recommended by the American Bridge Company, and also—in the author's opinion—agree with the practice of structural steel manufacturers in this country, and may therefore be adopted.

With regard to the maximum positions of the downpipes as found by calculation, the practice is to provide greater area than that calculated, therefore it is perhaps advisable to increase the area of the downpipes by about 33 %, or proportionately limit the distance between them.

GENERAL NOTES.

To prevent overflowing, gutters and pipes should be kept clear of debris, and for this purpose wire guards are often used to protect the discharge outlet from gutter to pipe. As a further precaution, it is good practice to keep the stop end of the gutter one to two inches below the level of the sides, to ensure any surplus water flowing off at the ends in preference to the sides.

To protect the gutter from snow, timber snowboards should be provided, resting on the slates at each side and immediately above the gutter. When such are provided they form a pathway along the roof, which gives a further protection to the gutter.

To facilitate the discharge from the gutter to the pipes, it is advisable to provide well-rounded corners to the outlets, or alternatively, and preferably, cesspools or enlarged pipe heads. Sharp bends to downpipes should be avoided, as they reduce the efficiency of the discharge and obstruct cleansing.

It is a common practice, especially for small span north-light roofs as used over weaving sheds, etc., to dispense with separate valley girders, by utilizing the valley gutters to support the roof rafters. This practice is inefficient and uneconomical, and should be avoided. It is better practice to use a thinner gutter and provide a steel H beam, the cost of the latter being amply covered by the saving in the weight of, and also in the cost of manufacturing, the heavier gutter.

Galvanized steel gutters are usually socket-jointed, the joint being made by means of a mixture of red or white lead putty. C.I. gutters are either socket or flange-jointed. Socket joints may be arranged with the socket projecting outside (the most common arrangement for the smaller sizes) or inside the gutter, the latter method, owing to a more or less permanent accumulation of water of a depth equal to the thickness of the obstruction, being used only where the outside socket would be an objectionable projection. The joint is made with red or white lead putty, to which—for the larger sizes—a layer of tow or other similar material is added. An alternative jointing material, which is undoubtedly the best for large gutters, is a mixture of sal-ammoniac and iron borings, the use of which makes a rough weld between the ends of the gutters. The proportions for the mixture are as follows :—

Quick-setting Cement.

1½% sal-ammoniac.
96% iron borings.
2½% flour of sulphur.

Slow-setting Cement.

1% sal ammoniac.
98½% iron borings.
½% flour of sulphur.

Flange joints are used for large gutters, and are superior to sockets. They increase the cost of manufacture, and are not generally used except for better class work. The flanges should be provided with facing strips, which may be either chipped level or machined, and the joint made with putty and tow or with the mixture prescribed above. All joints, whether of the socket or flange type, should be securely bolted together.

Large valley gutters, and also large boundary wall gutters, are often provided with ribs spanning across the waterway; that is, from side to side. They are intended to stiffen the sides, and as they obstruct the waterway, the efficient height must be taken as the distance between the underside of the rib and the bottom of the gutter.

Stop ends, outlets, returns, and other special parts of gutters may not be obtainable from stock, and will require to be specially manufactured, therefore where making-up lengths are required these special lengths may be utilized for this purpose.

Rainwater pipe heads, shoes, and bends may be obtained from stock. If specially ordered, then they may be used as making-up lengths, if desired.

Rainwater pipes have socket joints, which are made watertight by means of red or white lead putty similar to that used for gutters.

With regard to stock gutters, these may be obtained of different sizes to the same profile, and in at least two different thicknesses. Stock lists with prices are supplied by manufacturers, and, in addition, full size sections if specially applied for.

Rainwater pipes may be provided with shoes for discharging over an open gully trap, or may be finished plain and connected to the gully.

VENTILATION.

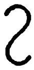
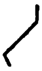
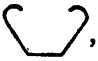
There are several arrangements for the ventilation of sheds and other similar buildings, but the fundamental principles may be narrowed down to two, i.e. natural and artificial. The natural system of ventilation is the one almost universally used for sheds, and the notes given hereafter will therefore be confined to this system. The object in view is that of removing—without setting up objectionable draughts—the vitiated air from the building and replacing it with fresh air.

The velocity and direction of the wind is the motive power whereby, in conjunction with the difference in the temperature between the air in and out of the building, this change of air is brought about; and as the wind acts generally in a horizontal direction, the best method of preserving its efficiency, whilst at the same time obtaining the best results, is to allow the wind to pass into and through the building without materially changing its direction. Openings should therefore be provided, transversely arranged at each side of the building, such openings being in such a position, as low down as possible—within, say, 3 to 5 feet of the ground-level—as to be unobstructed by adjacent structures. These openings should be small and frequent, so as to diffuse the incoming air over the interior of the building.

The amount of internal space per person to be allowed for in sheds is about 1400 cubic feet, and the amount of fresh air to be supplied about 5000 cubic feet, per person, per hour. In calculating the required air space, the efficient height should be taken as 12 to 15 feet above ground-level.

Fresh air is admitted at the lower levels, and diffused by means of special inlets, opening windows, and doors, and the total area of the inlets should equal, or slightly exceed, the total area of the outlets. The direction of the incoming air should be upwards, and the velocity, if excessive, should be decreased so as to prevent draughts. This influx of air will create a pressure inside the building, which, together with the ascending properties of the vitiated air, will accelerate the movement through the outlets. This movement may be further accelerated by means of specially constructed extractors, particulars of which may be obtained from the illustrated catalogues supplied by the manufacturers of such appliances, but except for small sheds this latter method is rarely adopted.

In arranging the relative areas of the inlets and outlets, it should be noted that, generally, the relative efficiency of the outlets will exceed that of the inlets, as the former are at a higher level, and consequently there is less obstruction from adjacent buildings. The outlet allowance per 100 square feet of floor space, as recommended in "Mill Building Construction" (H. G. Tyrrell), where louvre ventilators are used, is 4 to 9 feet, according to the nature of the building, the higher value being used for louvres at a height of 20 feet, and the lower value for a height of 50 feet, above ground.

The louvre ventilator is the one chiefly used for sheds, the profile of the louvre blades being ,  or , the first two being laid horizontally and the latter vertically. The horizontal type can be obtained from stock in lengths of about 6 feet. Louvre blades should be made of galvanized steel of a thickness of from Nos. 14 to 18 gauge, or from black steel sheets about $\frac{1}{8}''$ thick. They should be laid with ample laps, and secured to supports, at about 3 feet centres, by means of clips and bolts, and lap-jointed over alternate supports. Flashings should be provided between the roof and the top and bottom of the sides of the ventilator, although that at the top may be dispensed with if the roof of the ventilator is arranged to efficiently lap the top louvre blade. Louvre ventilators are not dustproof, and seldom proof against drifting snow, therefore if the nature of the use of the building is such that these disadvantages are not permissible, then the louvre blades must be arranged to open and close at will, or some other type adopted.

Where economy is of sufficient importance to exclude the use of louvres, a type of ventilator in common use, which, whilst cheaply constructed, is fairly efficient, is the "open" ventilator, where the opening between the roof sheets at the apex is protected by means of a curved sheet supported above, and from the roof, by means of suitable framework, the ends of the adjacent roof sheets being carried upwards to a radius of 12″ or more, so as to efficiently lap the overhanging curved sheet.

STEEL DOORS.

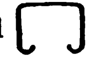
Doors hinged or sliding, constructed of steel angle framing with panels of corrugated sheeting, are used for openings up to 15′-0″ × 15′-0″, or even larger, but when these dimensions are exceeded, rolling shutters constructed of timber or steel laths may be used with advantage.

The sections generally used for the construction of steel doors panelled with corrugated sheets, whether of the hinged type or of the sliding type (if suspended from the top), are as follows :—

Height or width							
up to	8 ft.,	use	$2'' \times 2'' \times \frac{1}{4}''$	or	$2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ ∟	upright and rails	
8 ft. ,,	10 ,,	,,	$2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$	,,	$3'' \times 3'' \times \frac{1}{4}''$ ∟	,,	,,
10 ,, ,,	12 ,,	,,	$3'' \times 3'' \times \frac{1}{4}''$	,,	$3'' \times 3'' \times \frac{5}{16}''$ ∟	,,	,,
12 ,, ,,	15 ,,	,,	$3'' \times 3'' \times \frac{5}{16}''$	,,	$3'' \times 3'' \times \frac{3}{8}''$ ∟	,,	,,

Where necessary, crossbars of angle or tee section should be provided to limit the span of the sheets to 6′-0″, this crossbar to be of the same dimensions as the outer rail of the door. To prevent distortion of the rectangular frame, and in the case of hinged doors also to obviate the tendency to sag, flat ties should be provided extending diagonally from corner to corner of the door, the width and thickness of the ties being similar to the door upright. The direction of the bar is not an essential factor for sliding doors, but for hinged doors the top of the diagonal tie should commence from the top hinged corner.

The methods of hanging hinged doors may be narrowed down to two, i.e. by means of strong butt hinges, or by bands and gudgeons, the former arrangement being suitable for the smaller sizes and the latter arrangement for the larger sizes of doors. Fittings for hanging doors should be made very substantial, otherwise they soon wear and allow the door to sag, which necessitates constant alterations.

Sliding doors may be suspended from overhead runner bars, or be supported by runner bars at the bottom. Doors carried on bottom runners should be made of a heavier design than the suspended type of door. To reduce the effort required to start and keep the doors moving, frictionless rollers either of ordinary or patent pattern may be used with advantage—especially for large doors—in preference to the common roller and hanger where the pin works on a fixed centre. Patented designs for frictionless rollers are usually of the ball-bearing type, the roller path being formed of steel  section carried on special brackets. The rollers are connected to the hangers by means of a strong screw which allows the doors to be adjusted vertically. This type reduces frictional resistance to a minimum, and is very efficient, but owing to the cost being much greater than for the common type of frictionless roller—which reduces frictional resistance quite sufficiently for ordinary purposes—is seldom used, and only for first-class work. The feature of the common frictionless type is the formation of a roller path for the pin, the width of the hanger at this point being increased accordingly. The pin should be driven tight into the roller, and the ratio of the length of the path to the travel of the door should correspond with the ratio of the diameter of the pin to that of the roller, allowance being made, in addition, for the diameter of the pin.

The usual fittings for hinged doors are shootbolts, catches, hasps, staples, padlocks, and latches. Shootbolts are connected to the door by means of staples, and are provided to press the door against the door frame, or against catches arranged to prevent the door being swung too far, and also to secure and give rigidity to the door when closed. They are usually provided at the top and bottom of single doors, and also at the top and bottom of each half or both halves of a double door. If one half of a double door only is provided with shootbolts, then this half should overlap the other half, or some other device adopted so that the latter may be secured to the frame. Where contact with the frame is of sufficient importance, clips are provided for this purpose, at a point or points on the hinged side intermediate with the hinges.

To prevent doors being swung too far, if the door is not arranged so as to overlap the frame, then catches should be provided at the top and bottom of the door. The catch at the bottom of the door should be made so as to be flush with the floor when the door is open, and usually takes the form of a small bottomless box, the lid being made to open to an angle of 90° only. This box may act as the receptacle for the bottom shootbolt, providing the lid is perforated for this purpose. In large doors the bottom shootbolt is usually provided, at the top, with a specially-forged end, capable of engaging with an eye through which a padlock may be secured.

The latch at the centre of the side of the door opposite to the hinges should be provided with a suitable knob, and also with a catch firmly secured to the frame.

Sliding doors will require a track to run on, and also a guide to facilitate the opening. The track, whether at the top or bottom of the door, may be of angle, tee, channel, or joist section; flat bars are —in addition—used for over-tracks. If the track is overhead, the section used should have sufficient rigidity to efficiently span the opening with a minimum amount of deflection, and, if the roller is to run on the edge of the track flange, the latter should be planed to a square edge.

If the roller is intended to run on the flange of the runner-bar, then a small square bar should be countersunk-riveted to the flange. The lip of the roller should have a lap, over the runner-bar, greater than the space between the runner-bar and the top of the door; this precaution will prevent the door being jolted off the runner-bar. The bottom guide should be continuous, and of such a length as to always contain the bottom of the door. It may be of double angle section, the angles being held apart at a suitable distance by means of packings. The guide should be laid flush with the ground-level and firmly fixed to a suitable threshold of timber, concrete, or other material. If the door track is at the bottom of the door, then the foregoing remarks for overhead tracks will hold good with one or two exceptions. The lap between the door and the guide may, in both types, be arranged either by small angle projections or by a continuous angle secured to the door.

Handles for opening the doors, stops to prevent overrunning, hasps, staples, and padlocks for securing the doors are all the fittings required.

With regard to the corrugated sheets which form the panels, these should be bolted at intervals of 9" to 12" along the uprights, rails, and crossbars, and to each other at the joints, as described in the section of this chapter dealing with corrugated sheets.

WINDOWS AND SKYLIGHTS.

The order of efficiency in the transmission of light for various classes of glass are: British polished plate, sheet glass, rough cast glass, and rough rolled glass, the relative efficiencies being approximately 1, 1·5, 2, and 4.

Sheet glass is divided into five qualities, i.e. A, B, best, seconds, and thirds; seconds and thirds being used for sheds and similar buildings. It is usually specified by weight, 15 ozs., 21 ozs., 26 ozs., 32 ozs., 36 ozs., and 42 ozs. per super. foot, and the respective thicknesses are $\frac{1}{13}$", $\frac{1}{10}$", $\frac{1}{9}$", $\frac{1}{7}$", $\frac{1}{6}$", and $\frac{1}{5}$". For panes over 15" × 12" the thickness should not be less than $\frac{1}{9}$".

Rough-rolled plate glass, having one side fluted, is suitable for skylights, is specified by thickness, and the strengths in common use are: $\frac{1}{8}$", $\frac{3}{16}$", and $\frac{1}{4}$"; the respective weights per super. foot are approximately 32 ozs., 48 ozs., and 64 ozs., and the maximum size should—to facilitate delivery—be not more than 10 ft. × 3 ft. Wired glass may be obtained $\frac{3}{16}$" and $\frac{1}{4}$" thick, and may be used with advantage on account of its great strength, and also the fact that, in case of breakage, the fragments are held together and prevented from falling into the building.

The amount of glazing to be provided for varies considerably, according to the use of the building considered, but usually the limits are from 10% to 30% of the exterior surface of the building. The amount provided for the roof only may vary from 20% to 50% of the roof area, the higher figure being used for interior bays of sheds having several bays.

Windows may be constructed of timber, steel, wrought iron, or cast iron. If arranged to slide they are usually termed "sashes," and if pivoted are termed "casements."

Timber windows are usually constructed of red deal. For particulars as to design, reference should be made to a book on *Building Construction*, or the design may, with advantage, be left to the manufacturer.

Cast-iron windows are very little used at the present time, and steel windows have largely superseded wrought iron windows. The manufacture of steel windows, and also the opening gear, is a speciality with many firms, therefore the design should be left to the manufacturer, or a suitable type adopted from the trade catalogues. In fixing glass in steel or iron windows, the glass should be well bedded in red lead putty after the metal has been carefully primed.

If windows are used for sheeted walls, the sheets should be arranged so as to flash the joints between the windows and the framework at the sides and head, or the sides may be made watertight by means of a galvanized steel flashing. Generally the underside of the window is flashed by means of a galvanized steel sill, and if the windows are constructed of timber the sill may be hollowed out on the underside to a depth of ¾″ to 1″, and the sheet inserted.

Opening windows for use in tropical countries are often provided with mosquito guards constructed of galvanized iron or copper wire of fine mesh. The guards should be constructed in sections so that they may be easily moved for cleaning purposes, and with the edges suitably stiffened.

Ordinary skylights may be constructed of timber, or formed with small section tee bars spanning from purlin to purlin. For the design of timber-framed skylights, reference should be made to a suitable book on *Building Construction.*

Ordinary tee bar glazing costs very little less than patent glazing, and has therefore, to a large extent, gone out of favour. The bars are usually spaced as for patent glazing, i.e. 2′-0½″ centres, and the section, which varies according to the span, should be as follows :—

Up to 6′-0″ spans use 1¼″×1¼″×¼″ Tee.
6′-0″ „ 8′-0″ „ „ 1½″×1½″×¼″ Tee.
8′-0″ „ 10′-0″ „ „ 1¾″×1¾″×¼″ Tee.

Holes, through which pegs may be driven, are provided through the web of the tee at the top and bottom, and also in the centre, of long bars in a suitable position to allow the peg to bind on the glass. If necessary, the web at the top and bottom should be cut away for holing and flashing purposes, and at the bottom the flange of the tee is usually turned upwards, so as to act as a stop for the glass. If the tee astragal extends over two spans, then to save flashing the bar may be joggled at the centre, and the change in the level of the flange made to suit the thickness of the glass.

Patent puttyless glazing is usually constructed of steel bars either galvanized or covered with lead. There are many systems on the market, full particulars of which are given in the trade catalogues. The chief features are as follows :—

If the sashbar is of steel its cross section may be much less than that for a timber sashbar of equal strength, consequently the efficient area of skylight is greater.

The bar is either galvanized or completely clothed with lead, as a protection against the deteriorating effect of the atmosphere.

The glass is usually bedded on a cord of asbestos, and is held in place by means of the lead capping, or by a rigid zinc cap, in such a way as to allow the glass to expand or contract as the result of changes of temperature. The lead capping, and also the zinc cap, will prevent rattling due to vibration, but the zinc cap—which is secured to the steel bar by means of a stud tapped into the bar and screwed at the outer end, and provided with a nut—is the best arrangement in this respect. The use of the asbestos cord is to provide a compressible bearing for the glass, which would otherwise, if of uneven thickness, be liable to break if subject to excessive vibration.

All systems provide condensation channels, capable of carrying off moisture as the result of slight leakages or of condensation.

Broken panes can be easily replaced by unskilled labour, as the glass is simply inserted between the wings of the lead cap or underneath the zinc caps, as the case may be.

The initial cost of patent glazing is more than ordinary timber-framed or steel tee-bar glazing, but the greater efficiency obtained, together with the saving in maintenance cost, justifies its use.

The size of the steel bar is varied according to the span. The bars are secured to the purlins direct or by means of clips, and are also provided at the bottom with a brass stop.

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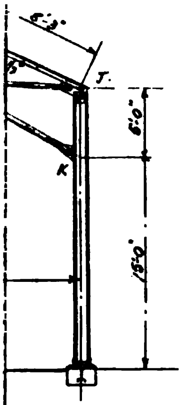
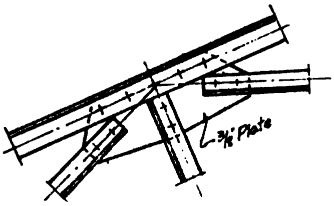
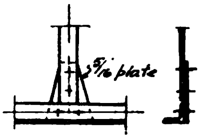
Plate $^{6}/L$







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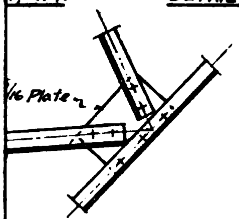

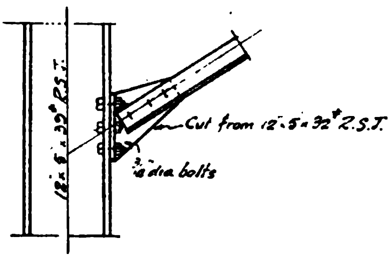

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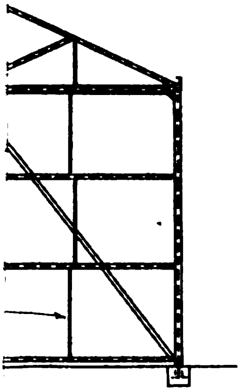

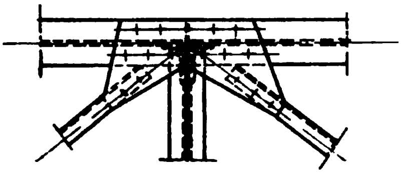

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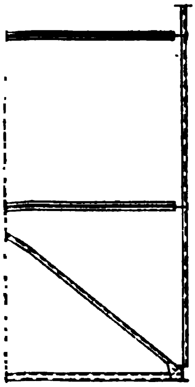

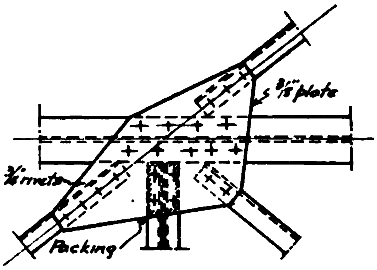

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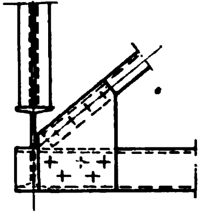

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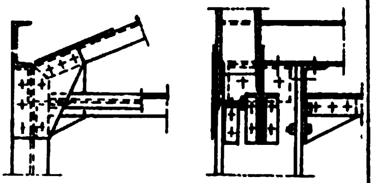

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